CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

AUGUST, 1953. **Vol. XLVIII, No. 8**

FORTY-EIGHTH YEAR OF PUBLICATION

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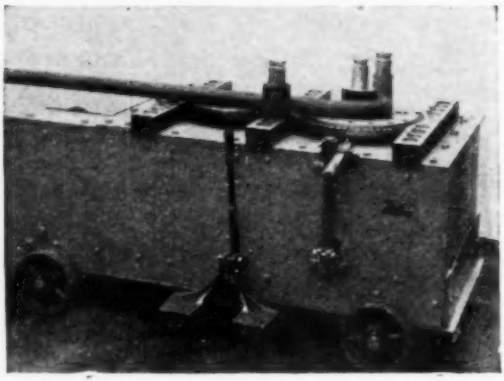

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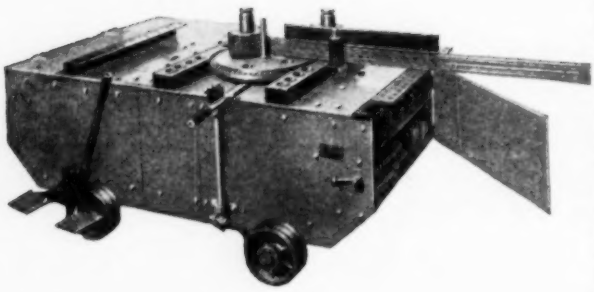

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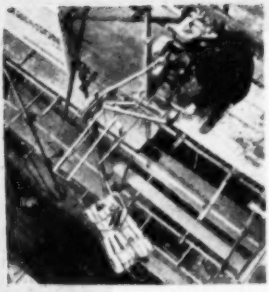

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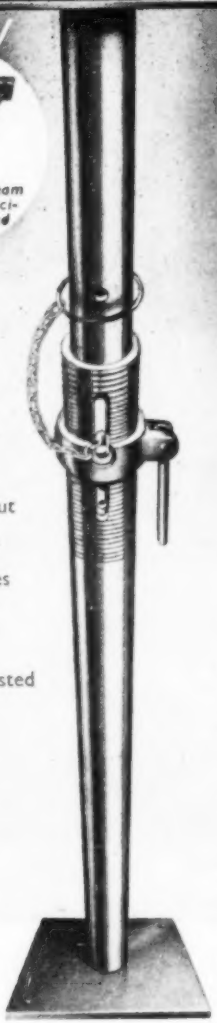

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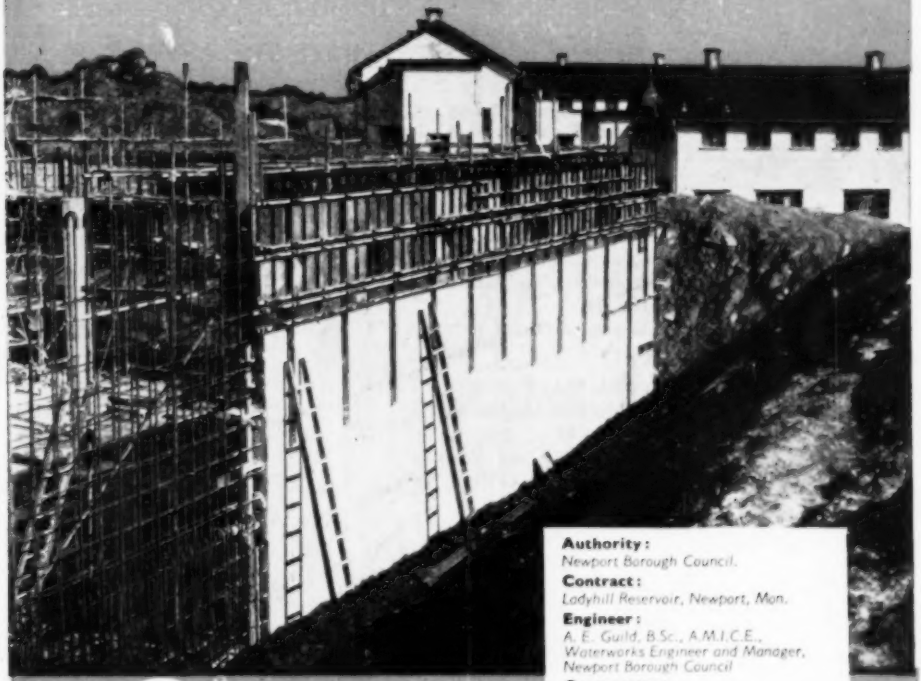

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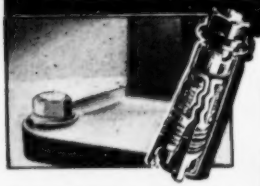

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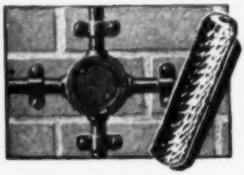

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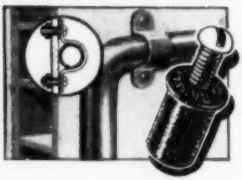

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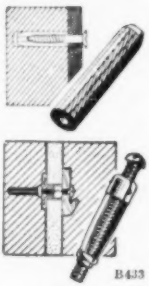

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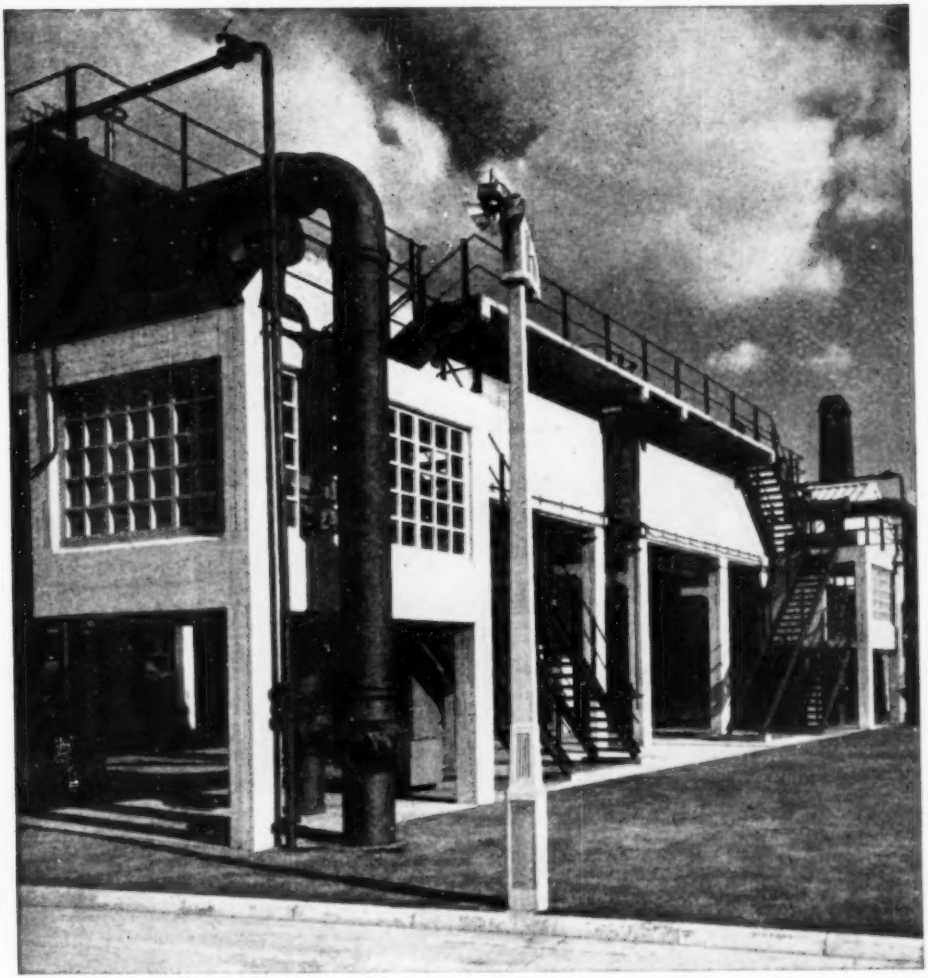

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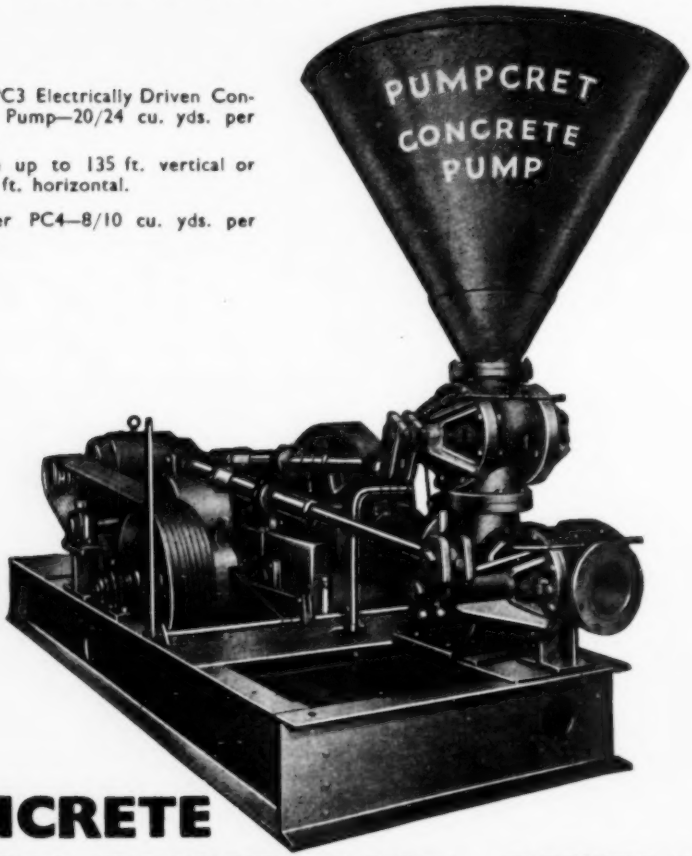

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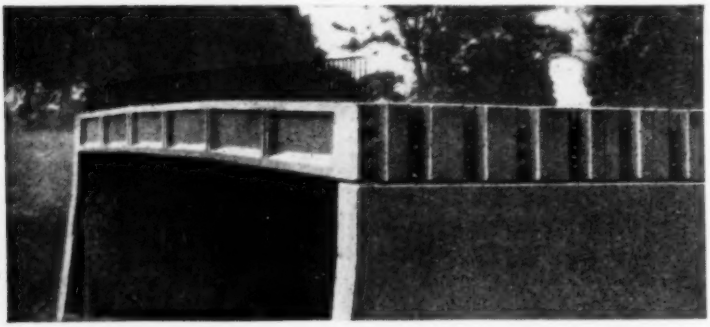

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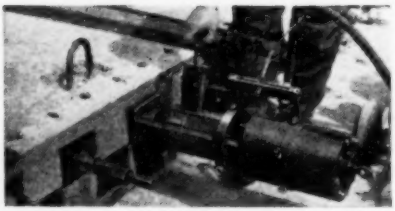

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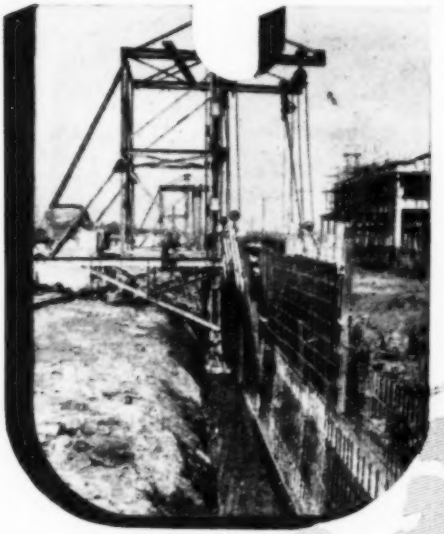

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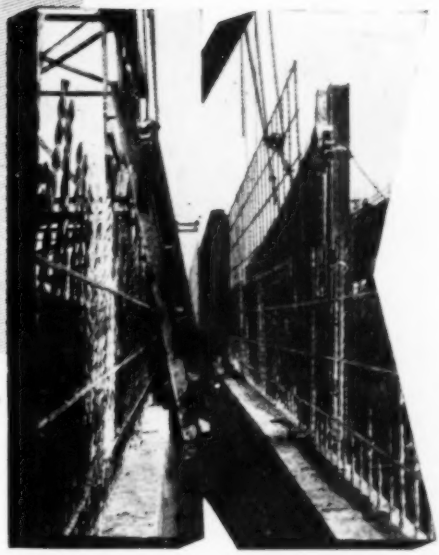

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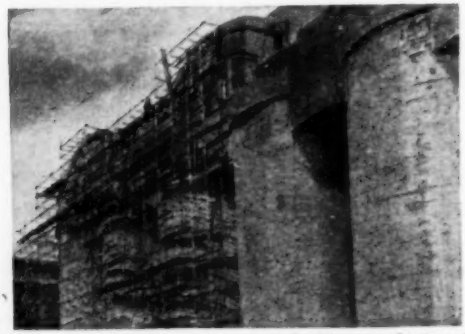

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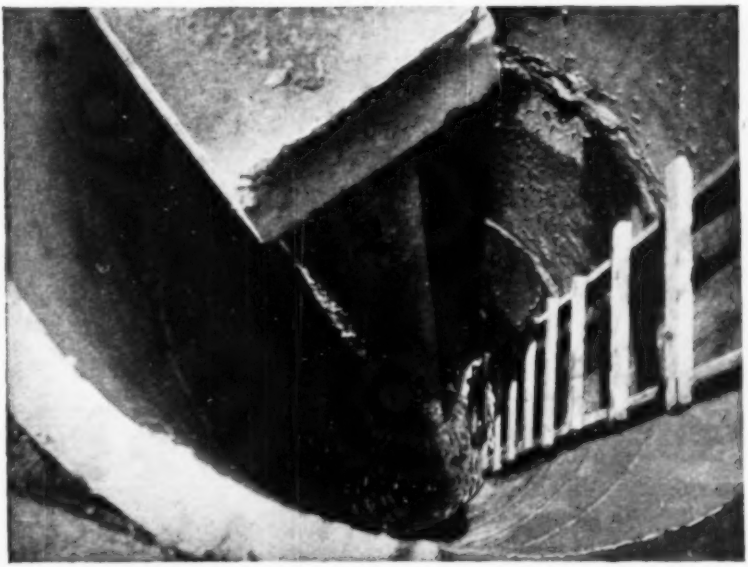

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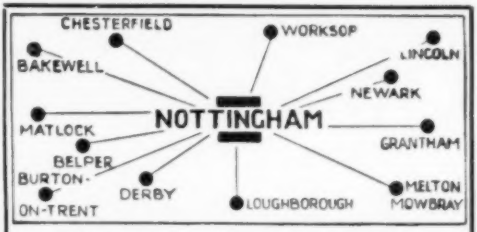

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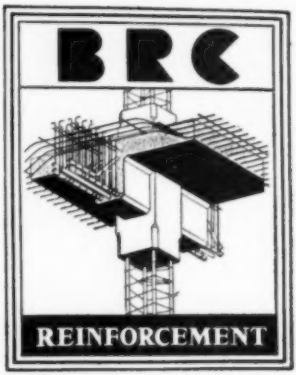

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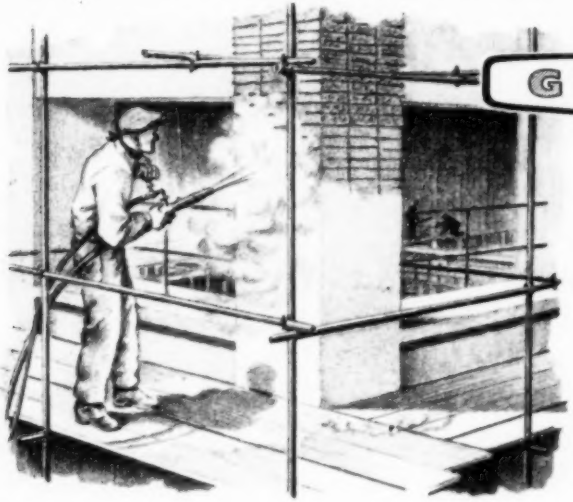

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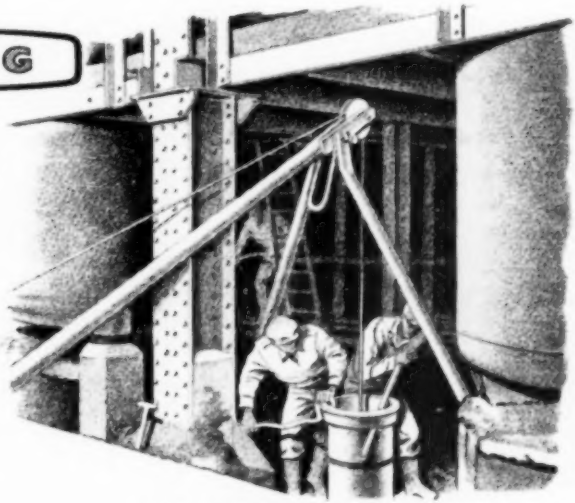

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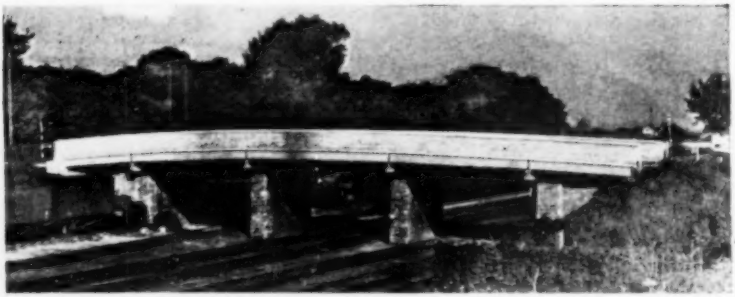

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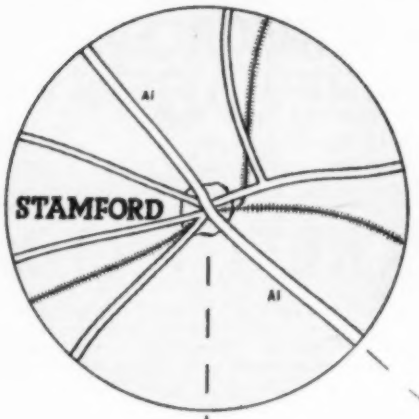

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Prestressed concrete beams in use on the partly-completed road-over-rail bridge on the Great North Road at Stamford. This illustration shows one of several trunk road bridges at present under construction using "ANGLIAN" prestressed beams.

Photograph by courtesy of British Railways, Eastern Region. Contractors: Messrs. Wellerman Bros. Ltd., Sheffield

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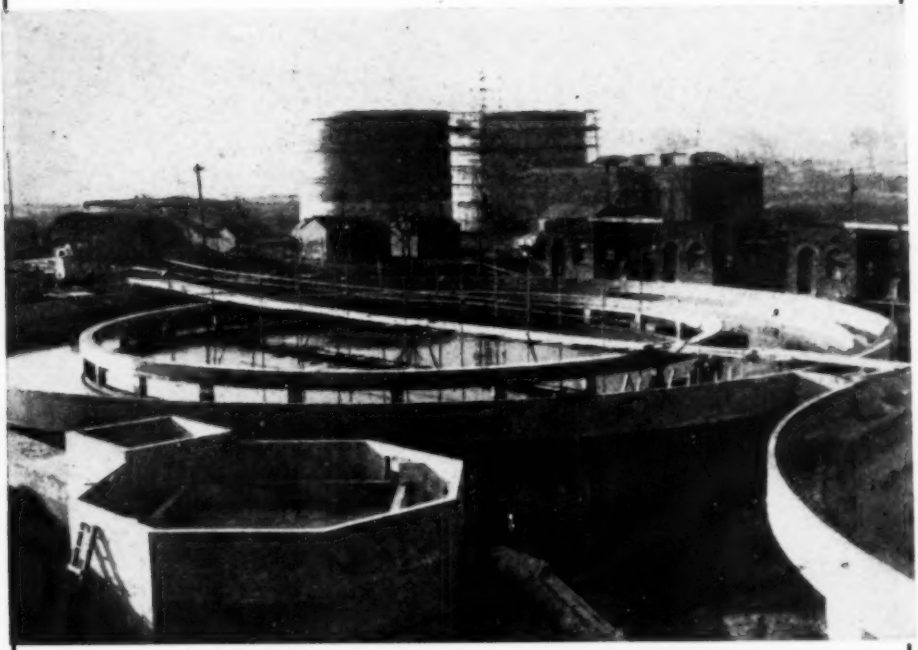

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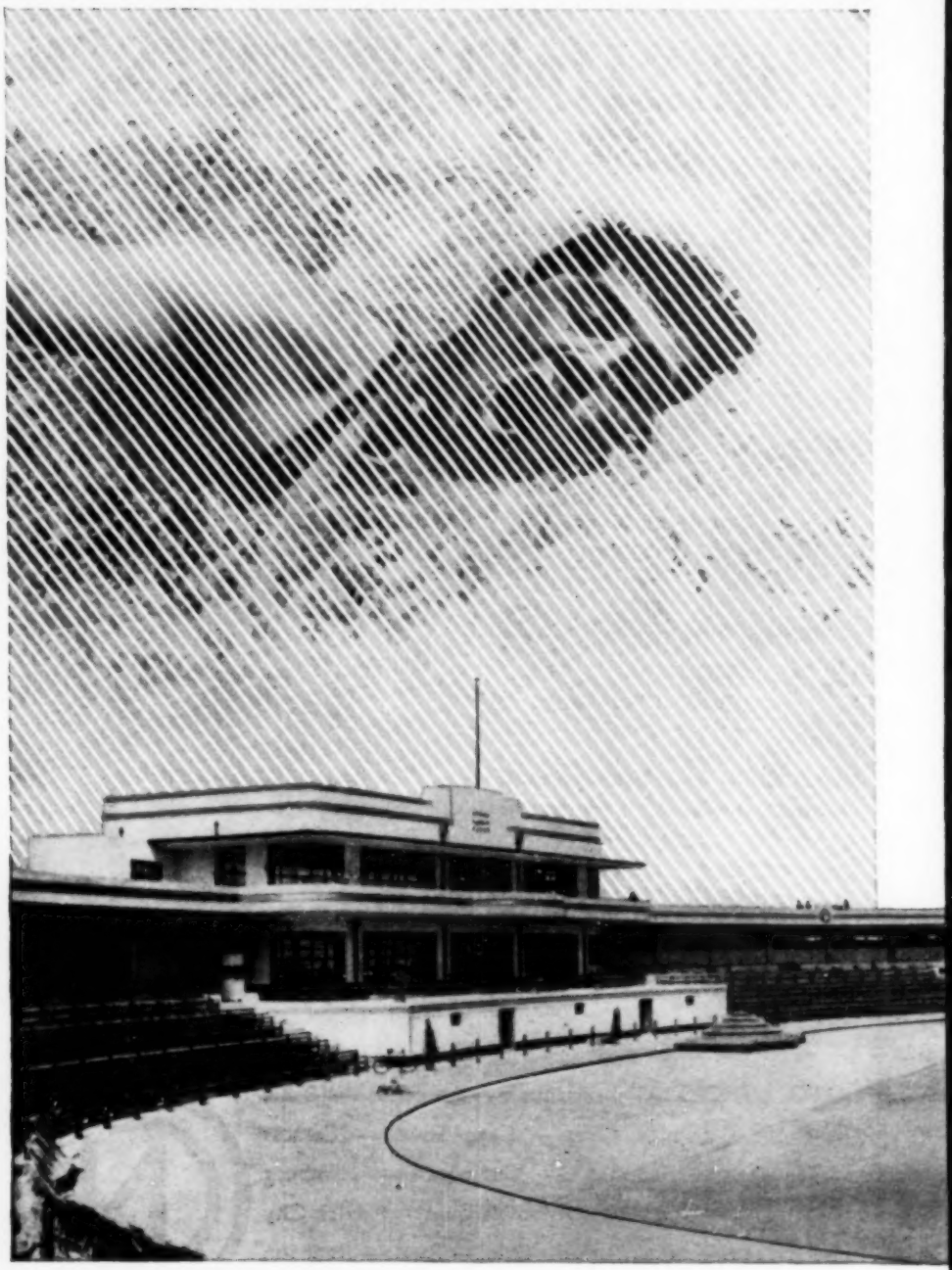

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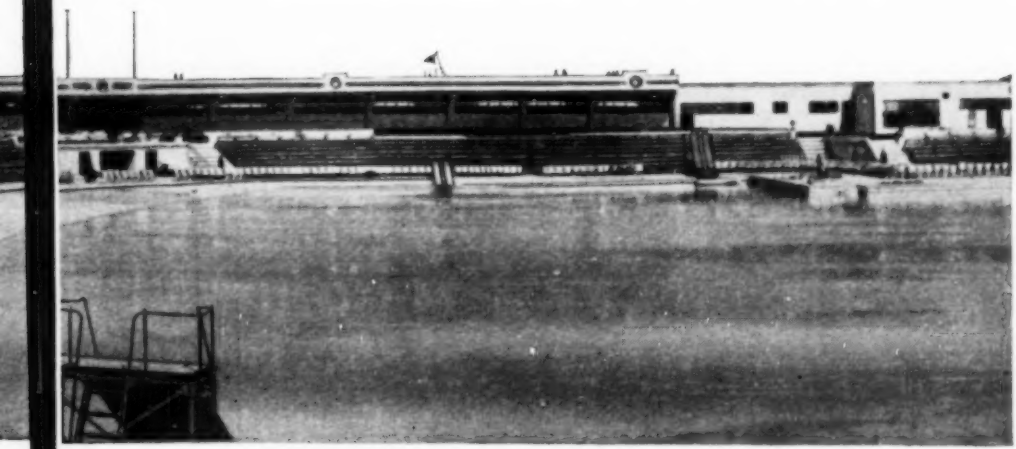

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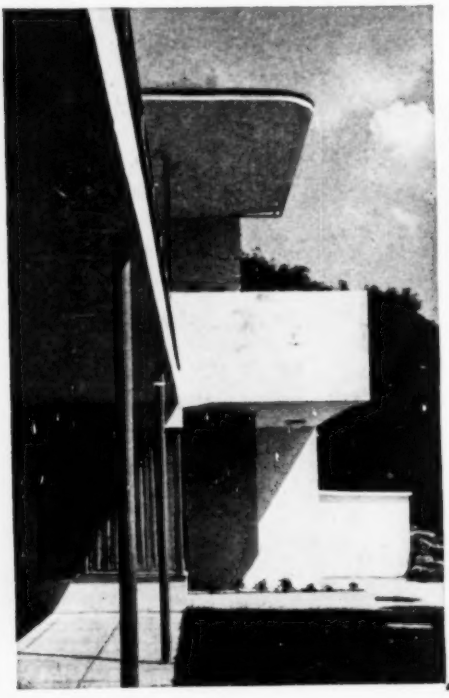

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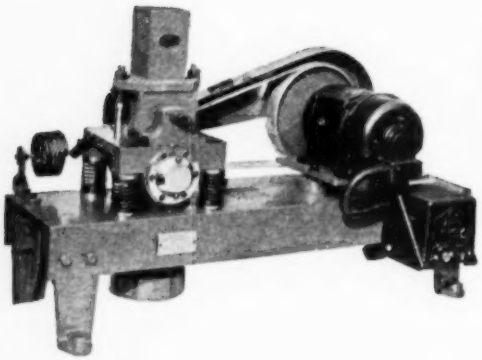

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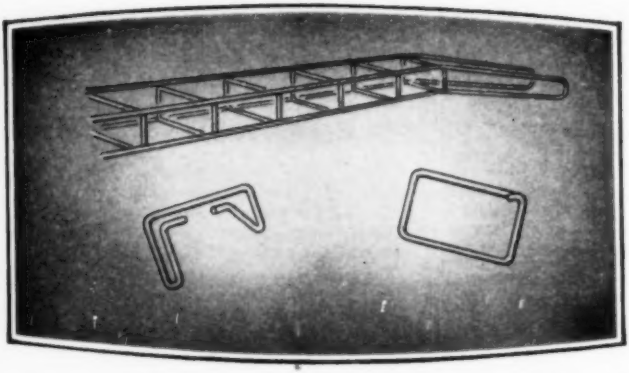

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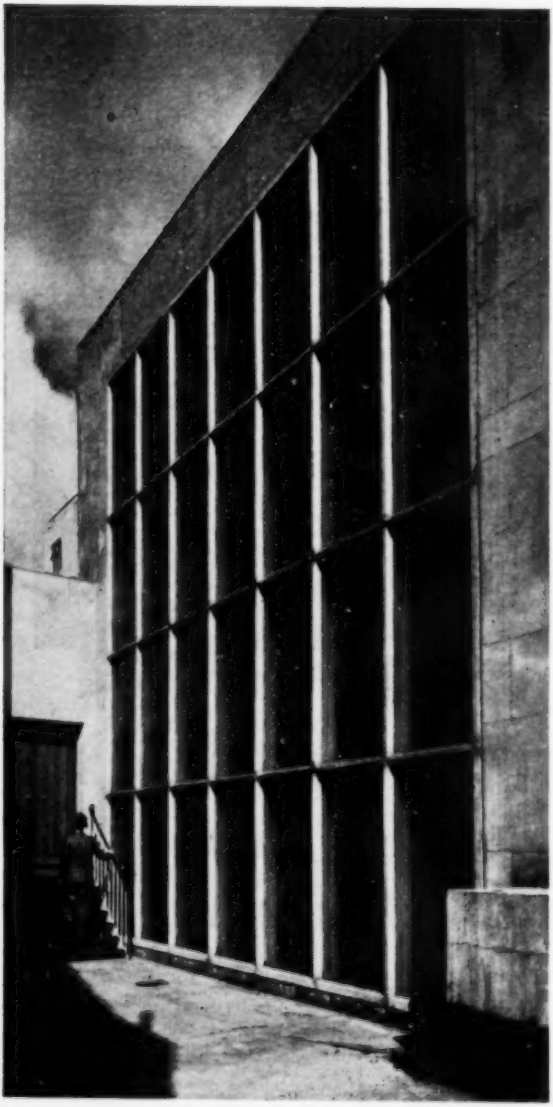

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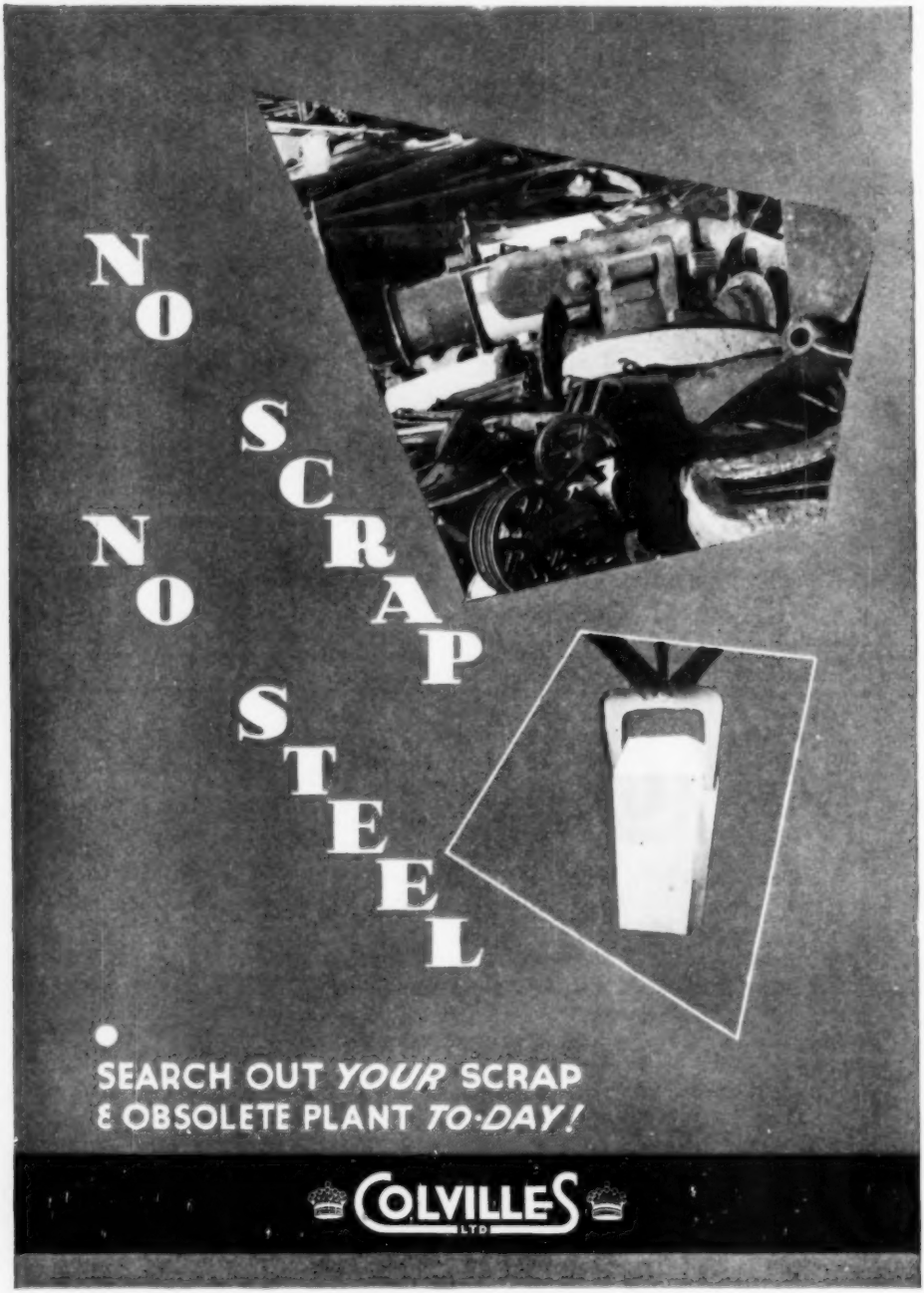

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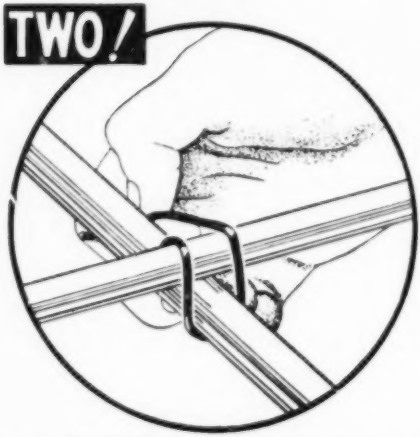

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The following is an extract from a letter received from a user of the C. & C.A. Covermeter regarding the illustration shown above—"I enclose a print of a reinforced concrete cantilever step that failed due to incorrect positioning of the reinforcing steel. Being one of a long double-flight staircase concern was felt for the safety of the other steps. The Covermeter was used, and they were all found to be unsafe in various degrees, necessitating cutting out and renewing. The new steps were checked by Covermeter before being built in."

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The Cement and Concrete Association Covermeter, illustrated above, has been specially produced to provide an easy, accurate, and economical means of finding the exact cover of reinforcement bars, or the position of prestressing cables, without interfering with the concrete and before the concrete has hardened. It thus enables Reinforced Concrete Engineers to make sure that the specified cover of reinforcement is maintained throughout the structure, and that the position of prestressing cables is in accordance with the design. It can also be used to find the position of reinforcement, gas pipes, and other fittings in existing structures. The C. & C.A. Covermeter can be supplied for Metric or Imperial systems of measurements. For full details and prices, send to the manufacturers:

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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume XLVIII, No. 8. LONDON, AUGUST, 1953.

EDITORIAL NOTES

Hydrophobic Cement.

A PORTLAND CEMENT that will not be affected by moisture or damp conditions during storage would be a boon to cement makers and users alike. In this journal for January, 1952, it was reported that such a cement was being produced in Russia, and in December last some details were given of a cement with hydrophobic properties that was patented in the year 1951. No details of the Russian product, the inventor of which was awarded a Stalin medal, are to our knowledge available in this country. Independent tests showed that the British product had another advantage besides its ability to resist moisture. It was found, for example, that prisms of mortar made with the hydrophobic cement had a greater resistance to the absorption of moisture than similar mortars made with ordinary Portland cement. Six-inch cubes made of 1 : 2 : 4 concrete had a density of 143 lb. per cubic foot and a crushing strength of 2390 lb. per square inch compared with a density of 147½ lb. per cubic foot and a compressive strength of 2540 lb. per square inch in the case of similar concrete made with ordinary Portland cement. The tests for permeability and compressive strength were made at fourteen days and seven days respectively after casting the test pieces. These results suggest that a decrease in permeability is counterbalanced by a decrease in compressive strength and in density. The results of tests at such early ages are, of course, of interest but of no real value to cement users, and engineers will look forward to the results of tests over longer periods which will no doubt be available later. Tests of the resistance to moisture of unset hydrophobic cement have, however, shown that it is not likely to be affected by poor storage conditions during the period which cement is generally kept on a site or in transit between the cement works and the site.

A short investigation of the subject has recently been made at the Building Research Station and the results confirm the claims made by the British and Russian inventors, although it must be emphasised that these were also short-period tests and are not a guarantee of the durability over a long period, which is essential in most constructional works, of concrete made with such cement. In the tests at the Building Research Station one of the chemicals used was oleic acid. The acid was ground into the cement clinker in proportions from 0·1 per cent. to 1 per cent. and a simple test of the permeability of the resultant cement was to see if it would float on water. Cement to which no oleic acid was added sank when it was sprinkled on to water ; with an addition of 0·1 per cent.

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of oleic acid the cement floated for seven days ; cements to which from 0·25 per cent. to 1 per cent. of oleic acid had been added all continued to float on the surface throughout the period of the test, and the report of the Building Research Station suggests that they would be likely to float indefinitely. Tests on 1 : 3 mortars with and without oleic acid showed that the initial and final setting times were 1 hour and 1 hour 10 minutes respectively for the treated cement compared with 2 hours 43 minutes and 3 hours 33 minutes in the case of similar cubes made without the additive. The expansion measured by the Le Chatelier test was the same, namely 1 millimetre, for the treated and untreated cements. Compressive tests on 1 : 3 mortar cubes up to 28 days showed that the addition of oleic acid retarded an increase of strength at very early ages, but that there was less difference at 28 days ; the results were as follows in pounds per square inch (the strengths of the cubes made with untreated cement are given in brackets) : One day, 316 (1090) ; three days, 2820 (3270) ; seven days, 5260 (5140) ; 28 days, 7090 (7600). It is suggested that the low strength of the treated cement at one day is probably due to the water-repellent film remaining on the particles of cement at that early period after casting.

A test on the effect of exposure to air gave some remarkable results. The test was made on 1 : 3 mortar cubes at the age of seven days, the cubes being made with ordinary Portland cement and with cement to which oleic acid had been added and both of which had been spread out in a layer half-an-inch thick and exposed to air at a temperature of 64 deg. Fahr. and at 90 per cent. relative humidity for periods up to six months, the cement being stirred and any lumps broken every four days. Cubes of 4-in. sides made with these cements without exposure had similar compressive strengths of about 5200 lb. per square inch. Cubes made with cements exposed for fourteen days had compressive strengths of 5100 lb. per square inch in the case of the treated cement and 2430 lb. per square inch in the case of the untreated cement. At three months the cubes made with untreated cement had no strength. The cubes made with treated cement after three months' exposure had a compressive strength of 5390 lb. per square inch, and cubes made with treated cement which had been exposed for six months had a compressive strength of 5800 lb. per square inch.

The addition of the chemical results in the formation of a water-repellent film around each grain of cement, which is rubbed off when the cement is subjected to abrasion with the aggregates during the mixing process. When the chemical is added to the cement clinker the grinding is facilitated with the result that a finer cement is produced. It would be expected that the finer cement would need more mixing water, but it is claimed that the chemical also acts as a plasticising agent and increases the workability of the concrete so that less water need be added to give a required workability, with the consequent possibility of increasing the strength of concrete with a given cement content or enabling a smaller cement content to be used. The chemical also entrains air in the mixture, and it was found at the Building Research Station that the air-entraining property of oleic acid is about the same as 0·1 per cent. of one of the air-entraining agents now available. Obviously there is no possibility of such cements being made commercially for years until experience has proved that there is no deterioration with age.

The Relative Economy of Framed Structures and Structures with Load-bearing Walls.

By N. E. COOPER, A.M.I.C.E., A.M.I.Struct.E.

CONFLICTING views are frequently expressed by architects, structural engineers, and quantity surveyors on the height at which load-bearing brickwork ceases to be economical. This is a complicated question and hard-and-fast rules cannot be laid down for all buildings. Many and varied factors must be considered. For example, the choice of contractor and the method of placing the contracts have a bearing on the matter; on large contracts it may, according to the experience of the main contractor, be better to have a main contractor and a specialist sub-contractor for the reinforced concrete work. In other cases it may be worth while designing a building as a framed structure rather than one with load-bearing brick walls for the sole reason that a specialist firm can be employed to erect the frame quickly and without having to wait for the general contractor to erect brickwork for each story. There are many other factors which directly affect a comparison between the methods which may be employed, for example the cost of the bricks, the proportion of facing bricks to backing bricks, the costs of aggregates, cement, and reinforcement, the load-bearing capacity of the ground, and the grid on which the spans will be determined, to name only a few. Nevertheless, in view of the recent shortage of steel, which now appears to be rapidly coming to an end, there were lately some surprising decisions made by architects in substituting load-bearing walls for columns in buildings of considerable height.

Nearly all architects and engineers are agreed that framed construction is structurally superior and preferable. A solid concrete slab is subject to shrinkage on hardening, and at all times to expansion and contraction due to variations in temperature, and these repeated movements cannot fail to affect the stability of any brick bearing-walls supporting the slab; for example, mortar joints are broken, bricks are cracked, and progressive deterioration from frost and weather is made easier. In a completely framed structure carrying walls at each floor level, trouble due to variations in temperature can be greatly reduced. It follows therefore that the choice between a framed building and one with load-bearing walls is largely a question of economy. In this review an examination is made of the most suitable form of construction for typical multiple-story flats where the greatest economy is essential.

Comparison of Three Possible Schemes.

It is generally agreed that for work in the London area at present the limit at which framed construction economically replaces load-bearing walls is somewhere between five and seven stories.

A scheme for a local authority in London for the construction of some blocks of flats, two of these being of five and six stories, and two of five stories only, for which tenders were submitted in the years 1949 and 1950, is the basis of the analysis given in the following. Two five-story blocks are examined in detail, and three possible methods of construction are considered, namely: (1) The building fully framed in reinforced concrete with 11-in. cavity walls, the whole weight of the building being carried, floor by floor, by the frame; (2) The building

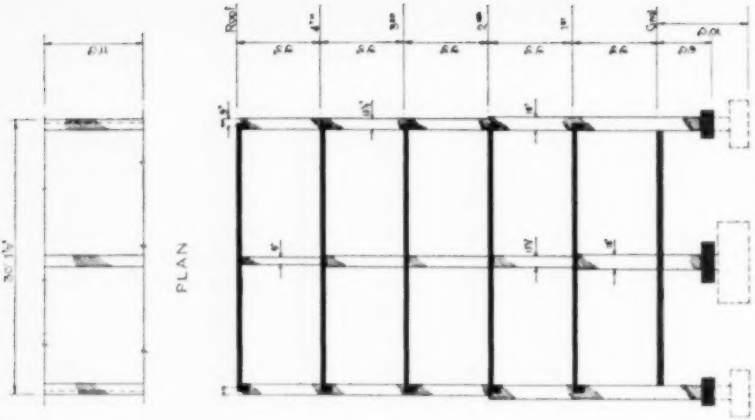


Fig. 3.—Schemes 3A and 3B.
Completely unframed.

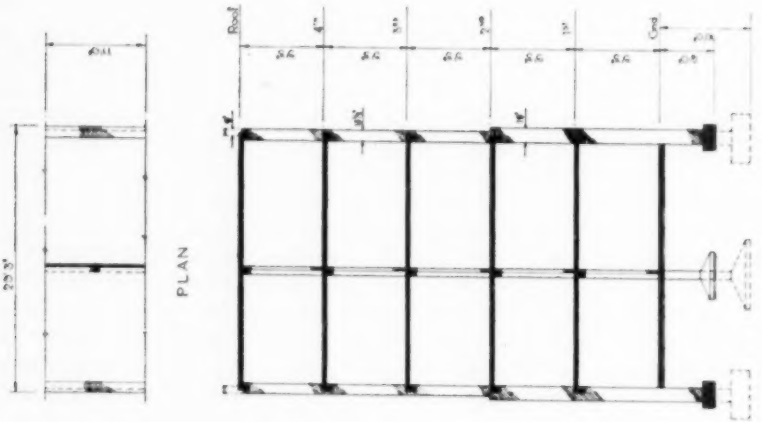


Fig. 2.—Schemes 2A and 2B.
Central spine of reinforced concrete beams and columns.

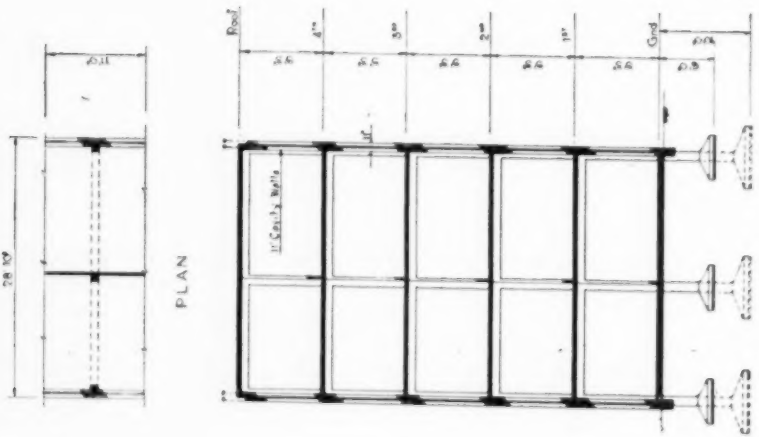


Fig. 1.—Schemes 1A and 1B.
Fully-framed.

carried by load-bearing external walls and a central spine of reinforced concrete beams and columns; and (3) The building carried entirely by load-bearing walls.

As a basis for comparing the schemes a typical bay of a five-story building 29 ft. wide by 11 ft. from centre-to-centre of the grid has been costed, as shown in *Figs.* 1, 2, and 3. The schemes are typical of those now being carried out in this country. The finishes to floors, walls, and ceilings are not included, neither is a ground-floor slab cast on the ground, since these are common to all three schemes and cost about the same in each case. The prices in the analysis, which were calculated from actual tender prices, therefore include only the reinforced concrete work, brickwork, excavation, foundations, and partitions.

It should be noted that the cost of dampcourses for the cavity walls at each floor level approximately equals the cost of the extra facing bricks needed for bonding the facing bricks into a solid wall. When load-bearing walls are used, however, the backing bricks are more expensive since much higher crushing loads have to be resisted.

Each scheme was further subdivided as follows: (A) Foundations designed for 2 tons per square foot and extending 6 ft. below ground-level; (B) Foundations designed for 1 ton per square foot and extending 10 ft. below ground-level; and (C) Foundations excluded altogether and the building considered from above ground-level only. These schemes are shown diagrammatically in *Figs.* 1, 2, and 3.

TABLE I.

COMPARATIVE COSTS OF AN 11-FT. BAY OF A FIVE-STORY BUILDING.

Scheme No. 1 (Fully framed)	Cost	Scheme No. 2 (Spine construction, i.e. partly framed)	Cost	Scheme No. 3 (Load-bearing walls only)	Cost
Estimate 1A	£861 (Fully framed. Foundations, 2 tons per sq. ft., 6 ft. deep)	Estimate 2A	£877 (Central spine. Foundations 2 tons per sq. ft., 6 ft. deep)	Estimate 3A	£969 (Load-bearing walls. Foundations 2 tons per sq. ft., 6 ft. deep)
Estimate 1B	£945 (Fully framed. Foundations, 1 ton per sq. ft., 10 ft. deep)	Estimate 2B	£997 (Central spine. Foundations 1 ton per sq. ft., 10 ft. deep)	Estimate 3B	£1142 (Load-bearing walls. Foundations 1 ton per sq. ft., 10 ft. deep)
Estimate 1C	£793 (As 1A, but excluding foundations)	Estimate 2C	£779 (As 2A, but excluding foundations)	Estimate 3C	£852 (As 3A, but excluding foundations)

The results of the estimates are as shown in *Table* I. The basic prices on which the costs were calculated represent prices current in London in 1949, and were as follows; all prices include delivery to the site: Facing bricks, per 1000, 185*s.*; Load-bearing backing bricks, per 1000, 150*s.*; Non-load-bearing backing

bricks, per 1000, 85*s.*; Cement, per ton, 71*s.* 6*d.*; Steel, basic price per ton, £22 3*s.* 5*d.*; Timber for shuttering, per standard, £70; Crushed stone aggregate ¾-in. gauge, per cu. yd., 15*s.* 2*d.*; Sand for concrete, per cu. yd., 17*s.*; Thames ballast for plain concrete, per cu. yd., 15*s.* 9*d.* The hourly wage rates were: Craftsmen, 2*s.* 10½*d.*; Labourers, 2*s.* 4½*d.*

Conclusions.

It will be seen that scheme 3 (load-bearing walls only) is very expensive and, in addition, there are many serious architectural disadvantages. This scheme may therefore be eliminated from further consideration. On the other hand,

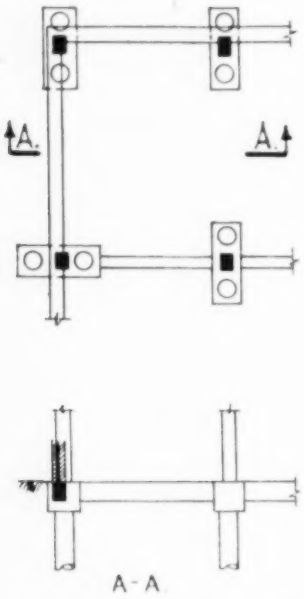


Fig. 4.—Piled Foundation for Fully-framed Building (Eight Piles).

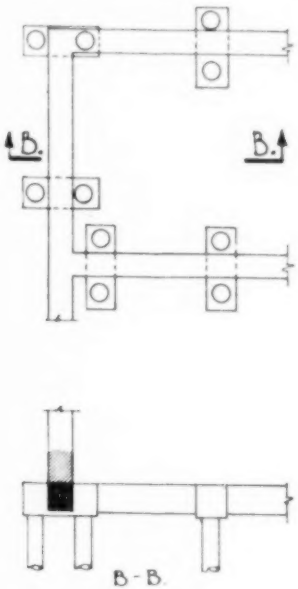


Fig. 5.—Piled Foundation for Load-bearing Walls (Ten Piles).

schemes Nos. 1 and 2 are very close in cost, which confirms that a five-story block is very nearly on the border-line where the costs of the two types of construction approach each other.

Examining schemes Nos. 1 and 2 in detail it may be noted that with foundations designed for a bearing pressure of 2 tons per square foot and at a depth of 6 ft. the cost is nearly the same for both schemes but very slightly (negligibly) favourable to the frame. For estimates 1B and 2B based on bearing pressures of 1 ton per square foot at a depth of 10 ft. the framed structure becomes markedly cheaper than the partly-framed one; this is to be expected in view of the very much heavier foundation for the load-bearing walls and the additional brickwork. Excluding foundations, however, in estimates 1C and 2C the balance reverts to the partly-framed scheme by a small but appreciable margin.

This comparison is most illuminating and illustrates what is so frequently forgotten in such comparisons, namely the great effect which the selection of the type of superstructure has on the cost of the foundation. It is quite common to prepare comparative costs for various types of superstructures on the incorrect assumption that the foundation is the same in each case, whereas in practice this is never so.

If the foundation has to be piled the difference in favour of scheme No. 1 is far more marked still, as is shown by *Figs.* 4 and 5. Not only is the number of piles increased considerably, but the ground-beams are very much larger and more expensive, having to carry a heavy load from above. Therefore for piled foundations the limit above which framed construction becomes economical may sometimes be as low as three stories.

It may therefore be concluded that for structures of contemporary design for domestic loads and with unpiled foundations on gravel and clay soil, with a bearing pressure of 2 tons per square foot, framed construction becomes economical from five stories upwards. In cases of piled foundations special calculations are necessary to determine the economical limit for a frame, but this will definitely be below five stories.

The Effect on Costs of Lightweight Walls.

It must be remembered that the foregoing calculations are based on 11-in. brick cavity walls. However, in accordance with the London Building Regulations which came into force on January 1, 1953, it is permissible to use a wall comprising a $4\frac{1}{2}$-in. brick outer leaf, a 2-in. cavity, and 3-in. clinker-block or hollow-block inner leaf. The effect of this tends to reduce the cost of framed construction very considerably but causes no corresponding reduction in the cost of load-bearing walls, and is therefore entirely in favour of framed construction. It follows that if an architect decides to use a light-weight inner leaf to external walls of cavity construction it is likely that the economical limit for unframed structures of the type considered is well below five stories.

The whole of the conclusions arising from this analysis apply, of course, with increased force in all cases where the foundation is designed for a bearing pressure of less than 2 tons per square foot. In other words, the more unreliable and yielding the ground the greater are the economies in the foundation due to framed construction.

International Federation of Prestressed Concrete.

For the purpose of co-ordinating the work of prestressed concrete associations in different countries, an International Federation of Prestressed Concrete has been formed. The office of the Federation is 28 Boulevard Raspail, Paris, and the secretary is M. J. Prempain. The Federation also has as its objects to promote the formation of national associations in different countries and to hold international congresses on the subject.

The first general meeting of the Federation will be held at the Institution of Civil Engineers, London, from October 6 to 9, 1953. Papers will be presented on The influence of abnormal temperatures; Design based on the ultimate load of statically-determinate beams and slabs; and Elastic and plastic phases in statically-indeterminate structures. Further information may be obtained from the Prestressed Concrete Development Group, Cement & Concrete Association, 52 Grosvenor Gardens, London, S.W.1.

Book Reviews.

"A Symposium on Prestressed Concrete Statically Indeterminate Structures." (London: Cement & Concrete Association. 1953. Price 25s.)

THIS contains the papers and discussions at the symposium held in London in September, 1951. The papers include: "Continuity of Prestressed Concrete Structures: the Practical Aspect," by Mr. A. J. Harris; "The Analysis of Statically-indeterminate Structures subjected to Prestress," by Mr. D. W. Cracknell and Mr. W. A. Knight; "Some Experimental Work on Interconnected Prestressed Beams," by Dr. P. B. Morice; "Continuity in Prestressed Concrete," by Professor G. Magnel; "Determination of Continuity Bending Moments in Prestressed Continuous Beams," by Mr. E. G. Trimble; "Continuity using Post-tensioned High-tensile Alloy Steel Bars," by Mr. G. O. Kee and Mr. S. Jampel; and "A Theoretical Treatment of Continuity in Prestressed Concrete," by M. Y. Guyon.

"Fahrbahnplatten von Strassenbrücken Berechnungstafeln." By Hubert Rüsch. (Berlin: Wilhelm Ernst & Sohn. 13 D.M.)

THE revised German specification D.I.N. No. 1075 dealing with road bridges requires deck slabs to be calculated in accordance with the theory of elastically-supported slabs. This book gives tables and diagrams in accordance with this theory for the design of the deck slabs of road bridges as required by the German regulations.

Data for the determination of the bending moments and reactions on slabs of various spans are given for uniformly-distributed loads and for concentrated loads ranging from 60-ton vehicles on trunk roads to vehicles weighing 3 tons on roads suitable for light traffic only. Compared with the Ministry of Transport loading for highway bridges this is a very flexible arrangement; it would, however, be interesting to know what steps are proposed to keep a heavy load off a road of lower classification.

In the case of long spans, a slab supported on two edges subjected to a load of 12 tons calculated by the old method produced bending moments in the principal direction five times greater than the bending moment derived from the elastic theory. This difference is attributed to the fact that the load distribution allowed for by the old method was inaccurate in the case of long spans. Only for spans below about 8 ft. 3 in. do the two methods approximate.

The tables are useful only in German practice so far as the disposition of the live load is concerned. The problem of load distribution has, however, been largely overcome in British practice by the introduction of a knife-edge load to allow for the concentration of the train of loads and of an equivalent uniformly-distributed load which decreases with increasing spans.

"Approximations Following from the Maximum Condition Applied in Theories of Plasticity and Earth Pressure." By H. Craemer. (Zurich: International Association for Bridge and Structural Engineering. 1952.)

METHODS of calculating the loads at failure of frameworks from the positions of plastic hinges have been developed for structural steelwork and to a lesser extent for reinforced concrete. Similar methods based upon the lines of fracture have been applied to reinforced concrete slabs by Dr. Johansen and Professor Craemer. In this paper, after a brief review of applications to rigid frames and slabs of constant thickness, the theory is extended to the calculation of earth pressures in granular soils.

"Lighting in Industry." (London: British Electrical Development Association. 1952. Price 8s. 6d.)

THE chapters are entitled, Lighting and Productivity, Lighting and Factory Managements, Some Particular Factory Lighting Applications, Lighting in Various Industries, Use and Maintenance of Factory Lighting, Colour in Factories, and Lighting Design.

"Specifications for the Construction of Housing Estate Roads Using Soil-Cement." Road Note No. 15. (London: H.M. Stationery Office. 1953. Price 9d.)

Two methods of constructing soil-cement roads, namely, the "mix-in-place" method and one using stationary mixers, are described in this pamphlet which has been prepared by the Soils Section of the Road Research Laboratory with the assistance of several engineers who have had experience of the processes. Although the specifications relate primarily to roads for light traffic the methods can be applied to the construction of roads to carry heavier traffic.

Prestressed Bridge across the River Rhine.

CANTILEVER CONSTRUCTION.

THE prestressed bridge shown in *Figs.* 1 and 2 has been built at Worms to replace a bridge destroyed in 1945. The bridge has three cantilever spans 333 ft., 375 ft., and 343 ft. long respectively, and was built without staging by Dyckerhoff & Widmann, K.G., on this firm's system of prestressing to a design by Dr.-Ing. H. Finsterwalder, chief engineer of the company.

To avoid the need for staging, the spans were cantilevered from two river piers (*Fig.* 3), from a pier on the left bank of the river (*Fig.* 4), and from an existing abutment on the right bank (*Fig.* 5). The cantilever on the left bank is counterbalanced by a girder near the tower of the old bridge. In the case of the cantilever on the right bank, the new construction would have strained the old foundation in the opposite direction to that of the previous bridge; a cantilever 120 ft. long was therefore built below ground level in order to counterbalance the structure. When the cantilevers were completed the ends were connected by hinges at the centres of the three spans. A longitudinal section through the bridge is shown in *Fig.* 6.

At the centre the beams have a depth of 8 ft. 2 in., increasing to 21 ft. at the supports. The spans comprise two U-shaped beams (*Fig.* 6) with a slab across them. Transverse girders occur only at the piers and at the centre of each span. In the remaining area the deck slab is prestressed both transversely and longitudinally. The deck is 45 ft. wide, comprising a road 25 ft. wide and footpaths and bicycle tracks each 5 ft. wide on both sides. The bridge was designed according to the German specifications to carry vehicles weighing 60 metric tons. For the cantilevers the concrete had a strength of not less than 6400 lb. per square inch; for the other members the strength was 4300 lb. per square inch. Sigma 60/90 steel was used for the prestressing bars of 1 in. diameter; this is a hard alloy steel with a well-defined

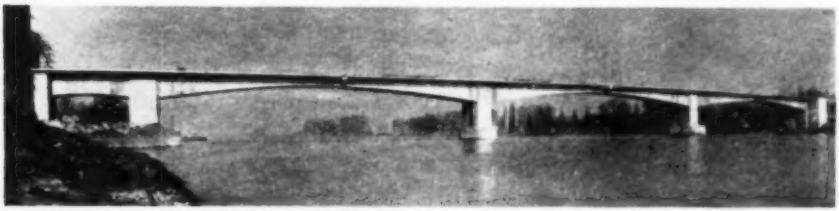


Fig. 1.—View from Left Bank.

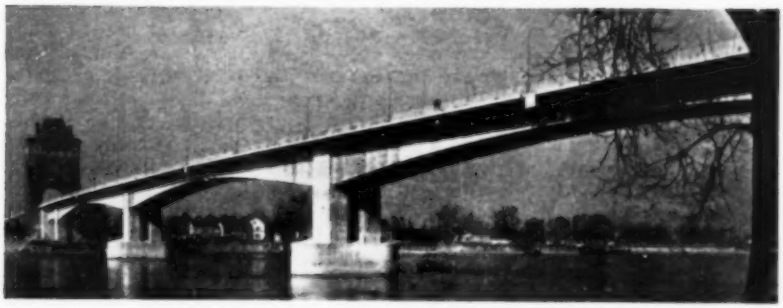


Fig. 2.—View from Right Bank.

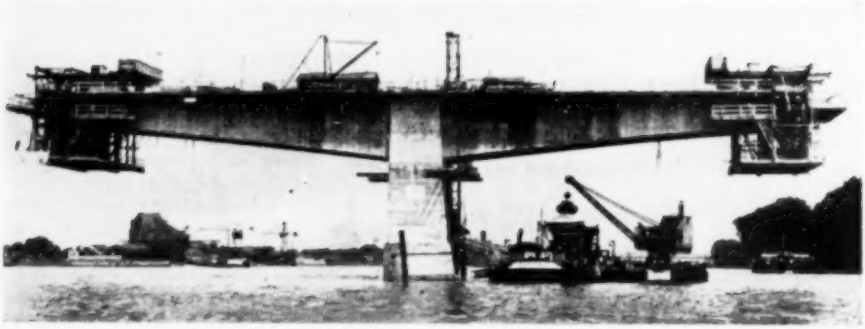


Fig. 3.—Construction from a Pier.

yield-point of about 92,000 lb. per square inch.

In this system of prestressing, the bars are anchored by steel plates and nuts. Both ends of the bars have cold-rolled threads, the work-hardening of the steel during the forming of the threads compensating for the reduction of the cross section due to the thread so that the core of the thread has the same strength as the rest of the bar. Couplings are used to join the bars as the work progresses. Each bar is encased in a thin-walled waterproof metal tube of 1¼ in. diameter to prevent bond between the bar and the concrete. After four days the bars are prestressed by an hydraulic jack, the nut at the end being continuously screwed on as the bar is lengthened. The extension of the bar is indicated to an accuracy of 0·004 in. by an instrument attached to the jack. After tensioning the bars the sheath is grouted. This system of construction has also been used for the construction of two other bridges of 200 ft. and 230 ft. span respectively.

The cantilevers in the bridge at Worms were built in sections 10 ft. long by means of a gantry travelling on rails. The gantry extended about 13 ft. over the completed part of the span. A working platform was suspended on the end of the gantry and on the completed beams of the previous section. The shuttering for the new section, consisting of wooden panels which could be used up to eighteen times, was supported by this platform. The frequent repetition of the same operations made it possible to divide the work into sections, namely (a) sliding forward and

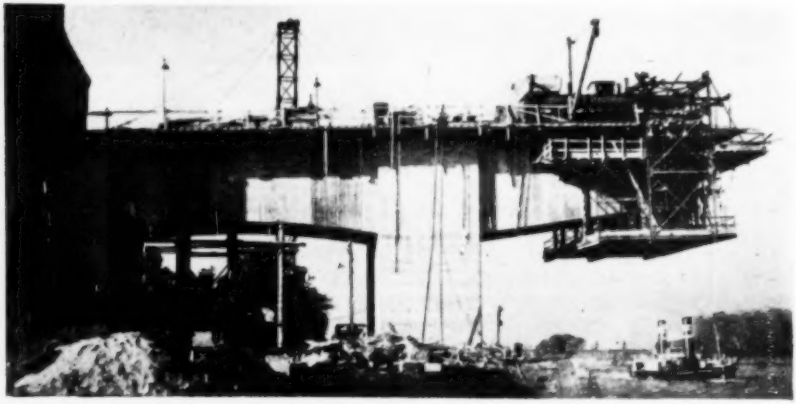


Fig. 4.—Construction from Left Bank.

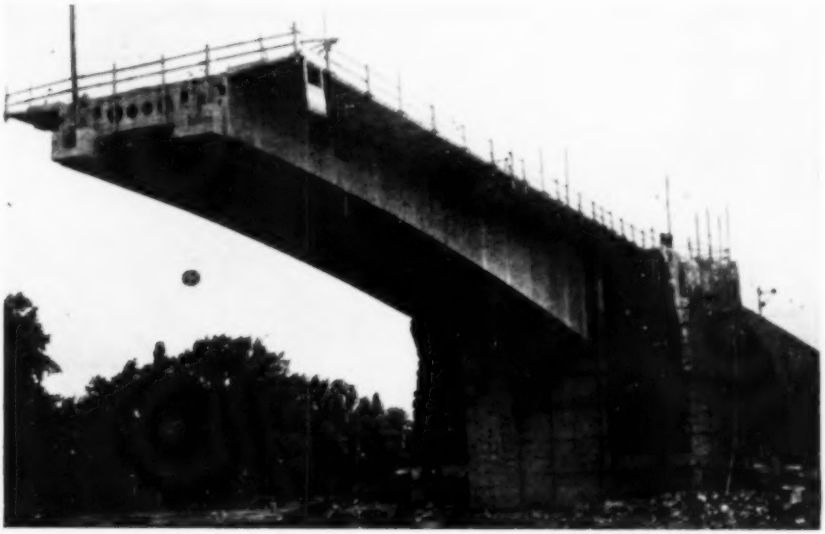


Fig. 5.—Beams Cantilevered from Abutment on Right Bank.

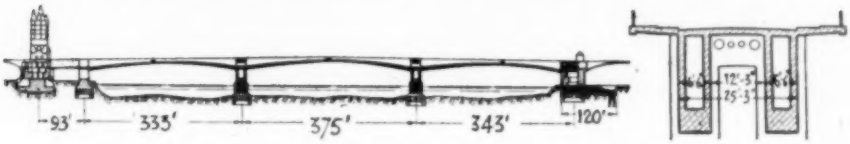


Fig. 6.—Longitudinal and Transverse Sections.

fixing the gantry, (b) fixing the shuttering, (c) fixing the sheaths and bars, (d) placing the concrete, (e) tensioning the bars anchored at the end of a new section, and (f) grouting the bars. The bars were arranged in groups of increasing lengths in 10-ft. increments so that a group could be anchored at the end of each section. The bars which were not anchored at a section remained loose in the sheaths until they were to be tensioned and anchored.

Construction started in the summer of 1951. First the piers and the counterbalance at the right-hand abutment were built. Construction of the spans started in March, 1952, and was completed in December, 1952. The new bridge (*Figs.* 1 and 2) was opened for traffic on April 30, 1953. Three gantries were in use at the same time. The cantilevers from the right-hand pier and from the right-hand abutment were completed first. The gantries were then transferred to the other river pier and to the abutment on the left bank of the river (the pier having been completed in the meantime) for use in the construction of the other spans. A total of 105 cantilevered sections of 10 ft. were each completed in an average working period of 7·9 days, including the time required to move the gantries from one position to another. The new construction comprises about 4250 cu. yd. of concrete, 365 tons of Sigma 60/90 steel, and 133 tons of mild steel reinforcement. The area of the bridge is 53,000 sq. ft. The spans contain 3 cu. ft. of concrete, 15 lb. of high-tensile steel, and 5½ lb. of mild steel per square foot of area.

The "Leoba" Method of Prestressing.

In "Beton-und Stahlbetonbau" for February, 1953, Dr. F. Leonhardt describes a method of prestressing known as the Leoba system. In this system flat cables, comprising twelve high-tensile wires of 0·2 in. diameter, are arranged in two layers of six wires. The wires are held in position by spacers (*Fig.* 1) and are enclosed in a metal sheath with a corrugated surface. The sheath is placed in the shutters and wired to mild steel distribution bars and stirrups. The high-tensile wires are loose inside the sheath which, due to its rectangular cross section, is flexible. One of the advantages claimed for the system is that as the sheath is easy to bend it can easily be adapted for continuous structures.

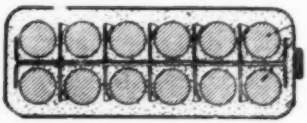


Fig. 1.—Cross Section through Cable.

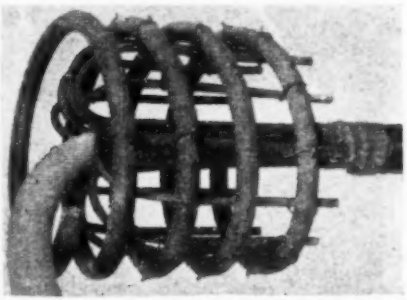


Fig. 2.—The Fixed End of the Cable.

Anchoring the Cables.

The tensioning end and the fixed end of the cable are constructed differently. Each wire has the shape of a hairpin. The ends of the wires, at the fixed end of the cable, are bent to form hooks (*Fig.* 2) forming an anchor which is strengthened by a mild steel helix. To avoid air-locks when the cable is grouted from the tensioning end a rubber hose is attached to the sheath at the fixed end of the cable and passed through the shutters. After the cable has been grouted the hose can be removed.

The tensioning end is shown in *Figs.* 3 and 4. The wires pass round a cast steel cross-head in the centre of which is a threaded hole in which a bolt is placed. The end of the sheath is formed into the shape of a funnel to accommodate the cross-head. This funnel-shaped end is surrounded by mild steel helices which resist the forces when the tensioned cable is anchored. The cable is placed in the shuttering before the concrete is cast and the tensioning end is held by a rubber sleeve (to the left of the cable in *Fig.* 4) placed in the enlarged end of the sheath. A bolt fixed to the cross-head passes through the sleeve and the shuttering and is retained by a nut inside the sleeve and a nut bearing against the outer face of the shutter.

When the concrete has hardened the nuts, sleeve, and bolt are removed and a threaded bar is screwed into the cross-head. A steel plate, through which the

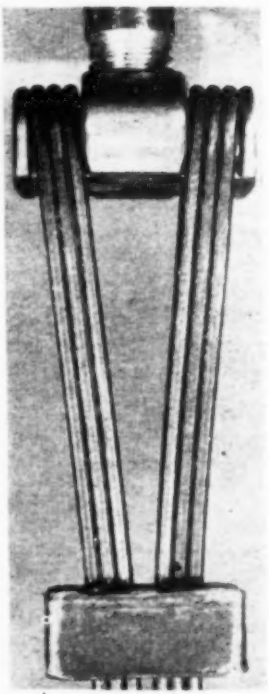


Fig. 3.—The Cross-head with the Wires.

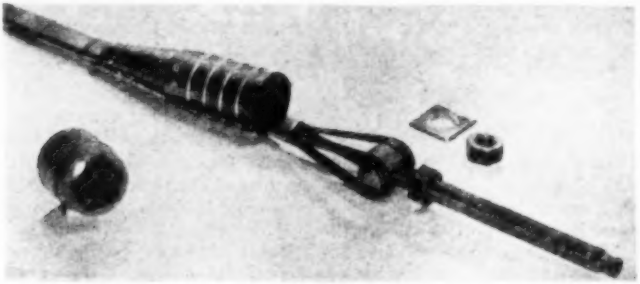


Fig. 4.—The Tensioning End of the Cable.

bar passes, is placed against the end of the concrete, and a rubber washer is placed around the bar and against the plate. A nut on the projecting part of the bar is tightened against the washer during the tensioning of the wires and provides an anchor when the jack is removed. The wires are tensioned by an hydraulic jack bearing against the steel plate at the end of the concrete, the ram of the jack being attached to the bar. The cable is grouted by means of a hose passing through a hole in the steel plate. When the grout has attained a sufficient strength the steel plate is removed and the protruding end of the bar cut off.

Tests to failure show that the wires break at the looped end and not at the hooked end. The forming of the loop also indicates the quality of the wires, because if this is not satisfactory they will break.

Friction in the Cables.

The frictional resistance of the wires depends on the condition of their surface and the water-tightness of the sheath which may allow a certain amount of cement grout to enter. If the wires are free from rust and no cement grout enters the sheath the frictional value is generally between 0·13 and 0·18. If the friction is greater it may be reduced by filling the cable with oil, but the oil must be removed before grouting, and if necessary the cable must be flushed with a solvent. If the frictional value is 0·18 or more it is advisable to increase temporarily the tensioning force by 15 per cent., and to reduce the force to its required value immediately before anchoring. A disadvantage of the system is that the cables can be tensioned from one end only, and to overcome this it is common to place the

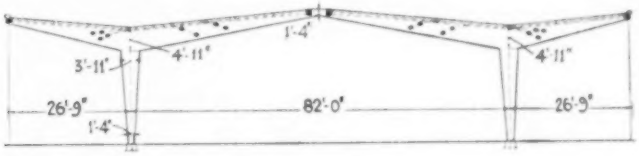


Fig. 5.—A Three-pin Frame.

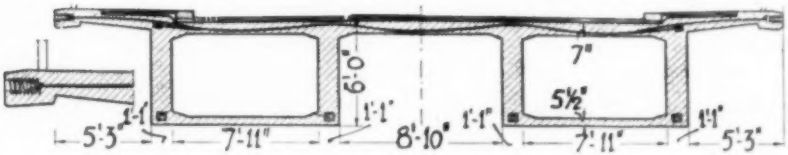


Fig. 6.—Cross Section through a Bridge.

cables with the tensioning ends alternately at opposite ends of the member. This also has the advantage that the tensioning ends are farther apart.

Applications.

Two structures in which this system has been used are shown in *Figs.* 5 and 6. A frame with a central span of 82 ft. and with cantilevers of 26 ft. 9 in. is shown in *Fig.* 5; the central span is hinged in the middle thus forming a three-pin frame. The method of anchoring the cables other than at the ends of a member is shown. The cross section of a bridge over the river Neckar is shown in *Fig.* 6. The cables are bent so that continuity is obtained. The deck is a slab of 36 ft. span continuous over four supports. The cables are arranged so that the fixed ends and the tensioning ends are alternately on the left or right side.—Abstracted by L. J. Elgin, A.M.I.C.E.

A Water Tower with Arched Walls.

THE water tower shown in *Fig.* 1 has been constructed in Egypt and was described in "Beton-und Stahlbetonbau" for December, 1952. The four raking columns support two tanks, placed one on top of the other and separated by a circular slab. Internally the tanks are cylindrical, but externally the wall is a series of arches with vertical abutments. The thrusts from the abutments, due to the pressure of the water, are resisted by the circular slabs at the base of the lower tank, between the tanks, and at the top of the upper tank. The arched slabs are also stiffened by cross-ribs.

It is stated that the advantage of this shape is that the water-retaining face of the wall is not subjected to tensile stresses and the possibility of cracks occurring is reduced.

Fig. 1.

The Structural Use of Steel Tubes.

BRITISH STANDARD CODE OF PRACTICE No. 113·201 (1953) entitled "The Structural Use of Steel Tubes in Buildings" has been issued (price 5*s.*) by the British Standards Institution. The Code gives recommendations for the design, fabrication, and erection of structures and independent members formed of steel tubes. Notes are included on the quality of materials, permissible working stresses, and methods of calculation. The code does not apply to scaffolding.

Analysis of a Parabolic Arch Supported by Columns Fixed at their Bases.—I.

By V. A. MORGAN, M.Eng., A.M.Inst.C.E.

THE moment of inertia of the arch rib to be analysed varies from crown to springings in the ratio of I_0 to $I_0 \sec\theta$, where I_0 is the moment of inertia at the crown. The frame is assumed to have the following dimensions, all given in terms of L, the span between the centre lines of the columns; the height (h) of column is L; the rise (r) of the centre line of the arch above the springings is $0{\cdot}5L$.

The equation of the centre line for the parabolic arch is

$$y = \frac{4r}{L^2}\left[\left(\frac{L}{2}\right)^2 - x^2\right] \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$\frac{dy}{dx} = \tan\theta = \frac{-8rx}{L^2}.$$

Therefore at the springings $x = \pm 0{\cdot}5L$, and since $r = 0{\cdot}5L$, $\tan\theta = \pm 2$, and $\sec\theta = \sqrt{5} = 2{\cdot}24$, so that the ratio of the moment of inertia at the springings

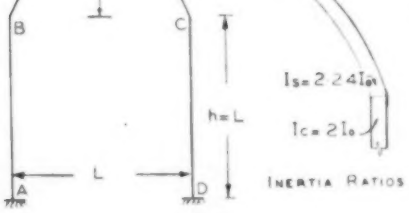


Fig. 1.

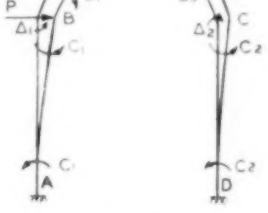


Fig. 2.

is to that at the crown as 2·24 is to unity. It is assumed that the moments of inertia of the columns are each twice that of the crown (*Fig.* 1). The stiffness K_s at springing B is equal to the moment at B causing unit rotation at B when springing C is fixed, and is $\frac{9EI_0}{L}$. Also K_B, the stiffness value of column B, is equal to the moment at B causing unit rotation at B when base A is fixed, and is $\frac{4EI_c}{L}\left(=\frac{8EI_0}{L}\right)$. Thus the ratio of the stiffness of the arch to that of the columns at B and C is as 9 is to 8. The carrying-over factor, as shown in previous articles,[1] is minus one-third.

Horizontal Load Acting at the Springings.

The horizontal load P acting at B causes deflections Δ_1 at B and Δ_2 at C. Anticlockwise moments are assumed to be positive.

[1] "An Exact Method of Analysing Symmetrical Parabolic Arches." October, November and December, 1947.

TABLE I.

CARRY OVER FACTORS:–	+½		−⅓		+½
A		B		C	D
1	STIFFNESS RATIOS	8	9	9	8	1
C_1	FIXING MOMENTS	C_1	$-D_1$	D_1	C_2	C_2
		$^{8}/_{17}(D_1-C_1)$	$^{8}/_{17}(D_1-C_1)$	$-^{8}/_{17}(D_1+C_2)$	$-^{8}/_{17}(D_1+C_2)$	
$^{4}/_{17}(D_1-C_1)$		$^{3}/_{17}(D_1+C_2)$	$-^{3}/_{17}(D_1-C_1)$		$-^{4}/_{17}(D_1+C_2)$	
	$-^{24}/_{17^2}(D_1+C_2)$	$-^{27}/_{17^2}(D_1+C_2)$	$^{27}/_{17^2}(D_1-C_1)$	$^{24}/_{17^2}(D_1-C_1)$		
$-^{12}/_{17^2}(D_1+C_2)$		$-^{9}/_{17^2}(D_1-C_1)$	$^{9}/_{17^2}(D_1+C_2)$		$^{12}/_{17^2}(D_1-C_1)$	
	$^{72}/_{17^3}(D_1-C_1)$	ETC.				

FIXED-END MOMENTS.—The fixed-end moments at the top and bottom of column AB $= C_1 = \dfrac{6I_c\Delta_1 E}{h^2}$; at the top and bottom of column CD the fixed-end moments $= C_2 = \dfrac{6I_c\Delta_2 E}{h^2}$. The fixed-end moments for the arch are $-D_1$ at B and $+D_1$ at C, where $D_1 = \dfrac{15EI_0(\Delta_1-\Delta_2)}{2Lr}$. The deflections and fixed-end moments are shown in *Fig.* 2 and the moments are distributed as in *Table* I. It is seen that alternate distributions of these moments decrease in geometrical progression with a ratio of $\dfrac{9}{289}$, so that their sum to infinity is their first term multiplied by $\dfrac{289}{280}$. Thus

$$M_{BA} = C_1 + \left[\frac{8}{17}(D_1 - C_1) - \frac{24}{17^2}(D_1 + C_1)\right]\frac{289}{280} = \frac{18C_1 - 3C_2 + 14D_1}{35}.$$

Substituting for Δ_1 and Δ_2 in terms of C_1 and C_2, $D_1 = \dfrac{5}{4}(C_1 - C_2)$ so that

$$M_{BA} = \frac{71C_1 - 41C_2}{70}, \quad M_{CD} = \frac{71C_2 - 41C_1}{70}, \quad M_{AB} = \frac{141C_1 - 41C_2}{140},$$

and $M_{DC} = \dfrac{141C_2 - 41C_1}{140}$.

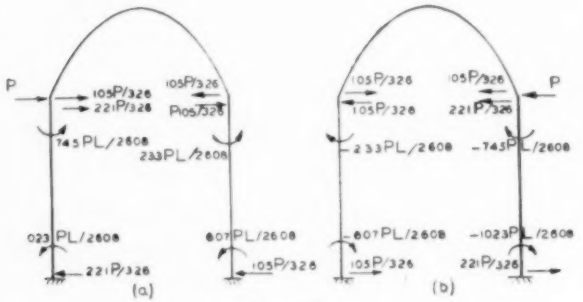


Fig. 3.

From previous articles in this journal,[2] $H_{arch} = H_0 + \frac{5}{8r}(M_{BC} - M_{CB})$, where H_0 is the thrust when the springings are hinged. Also, $M_{BC} = -M_{BA}$, and $M_{CB} = -M_{CD}$.

$$H_0 = \frac{-15EI_0}{8r^2L}(A_1 - A_2) = \frac{-5}{8L}(C_1 - C_2) \text{ so that } H_{arch} = \frac{-21}{8L}(C_1 - C_2)$$

(the minus sign denotes inward thrust at the springings).

This thrust must equal the shear H_2 on column CD where $LH_2 = M_{CD} + M_{DC}$, hence

$$\frac{21}{8L}(C_1 - C_2) = \frac{283C_2 - 123C_1}{140L}, \quad C_1 = \frac{1301C_2}{981},$$

and

$$H_1 = \frac{283C_1 - 123C_2}{140L} = \frac{1768C_2}{981L}, \quad H_2 = \frac{840C_2}{981L},$$

and

$$P = H_1 + H_2 = \frac{2608C_2}{981L}.$$

Therefore, substituting for C_2 and C_1 in terms of P [*Figs.* 3(*a*) and (*b*)]

$$H_2 = \frac{105P}{326}, \text{ and } H_1 = \frac{221P}{326}.$$

$$M_{BA} = \frac{745PL}{2608}, \quad M_{AB} = \frac{1023PL}{2608}, \quad M_{CD} = \frac{233PL}{2608}, \quad M_{DC} = \frac{607PL}{2608}.$$

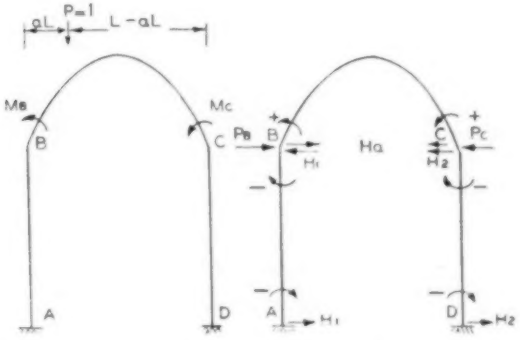


Fig. 4. **Fig. 5.**

TABLE II.

A ½	8 B 9 −⅓	9 C 8 ½ D
	Ma	Mc
−⅝ Ma	−⅝ Ma	−⅝ Mc −⅝ Mc
−⅘ Ma	⅗ Mc	⅗ Ma −⅘ Mc
	ETC	

[2] "An Exact Method of Analysis of Continuing Parabolic Arches." November, 1952, and March, 1953.

Influence Lines for Bending Moments due to a Vertical Unit Load Moving Across the Span.

CASE 1.—If joints B and C are prevented from deflecting the fixed-end moments at B and C due to a unit load on the span are (*Fig.* 4),

$$M_B = \frac{La}{2}(1-a)^2(2-5a), \text{ and } M_C = \frac{La^2}{2}(1-a)(3-5a).$$

Distributing these moments as in *Table* II we obtain

$$M_{BA} = -\frac{1}{35}(17M_B + 3M_C), \quad M_{AB} = 0{\cdot}5\ M_{BA},$$

$$M_{CD} = -\frac{1}{35}(17M_C + 3M_B), \quad M_{DC} = 0{\cdot}5\ M_{CD},$$

$$M_{BC} = -M_{BA}, \text{ and } M_{CB} = -M_{CD}.$$

Let H_a be the thrust contributed by the arch, then $H_a = H_0 + \frac{5}{8r}(M_{BC} - M_{CB})$.

From previous articles $H_0 = -\frac{5}{8}\frac{L}{r}a(1-a)(1+a-a^2)$, then

$$H_a = H_0 - \frac{1}{2L}(M_C - M_B) = -\tfrac{3}{4}a(1-a)(1+5a-5a^2),$$

the negative sign indicating an inward thrust.

$$\text{As } LH_1 + M_{BA} + M_{AB} = 0,\ H_1 = \frac{3a(1-a)(17-55a+35a^2)}{70}.$$

Similarly $$H_2 = \frac{3a(1-a)(3+15a-35a^2)}{70}.$$

If a is less than $\frac{2}{5}$, M_B and M_C will both be positive and the directions of the moments will be as in *Fig.* 5. Then the resultant force which acts on an assumed support at B is

$$P_B = H_{aB} - H_1 = \frac{a(1-a)(3+855a-735a^2)}{140}.$$

This acts towards the right, because H_{aB} is greater than H_1. Similarly the resultant force on a virtual support at C is

$$P_C = H_{aC} + H_2 = \frac{a(1-a)(123+615a-735a^2)}{140}.$$

As both H_2 and H_{aC} act to the left, the resultant P_C must act towards the left.

CASE 2.—The assumed supports are now removed from B and C and the joints are allowed to deflect under the action of the thrusts. The forces H_1, etc., are resisted by forces equal and opposite to P_B and P_C due to the rigidity of the frame. The components of these forces are added to H_1, etc., in directions opposite to those shown in *Fig.* 3(*a*) for the components of P_B and in directions opposite to those in *Fig.* 3(*b*) for the components of P_C. Thus the final value of the thrust is $H_1 + \frac{221P_B}{326} - \frac{105P_C}{326}$, and after substitution and reduction,

$$H_{1\,final} = \frac{15a(1-a)(5+4a-4a^2)}{163}$$

acting to the right at B and to the left at A. Similarly at C,

$$H_{2\,final} = H_2 + \frac{105P_B}{326} - \frac{221P_C}{326} = \frac{-15a(1-a)(5+4a-4a^2)}{163},$$

the negative sign indicating that the force acts to the left.

As a check, it will be found that $-H_{aB} + \frac{105P_B}{326} + \frac{105P_C}{326} = -H_{1\,final}$.

$$M_{BA\,final} = M_{BA(case\ 1)} - \frac{745P_BL}{2608} + \frac{233P_CL}{2608}, \text{ and as}$$

$$M_{BA(case\ 1)} = -\frac{L}{70}a(1-a)(34-110a+70a^2),$$

therefore,
$$M_{BA\,final} = \frac{-a(1-a)(539+286a+40a^2)L}{1304}.$$

Similarly
$$M_{AB\,final} = \frac{-a(1-a)(61-766a+440a^2)L}{1304},$$

$$M_{CD\,final} = \frac{a(1-a)(213+366a-40a^2)L}{1304}, \text{ and}$$

$$M_{DC\,final} = \frac{a(1-a)(387+114a-440a^2)L}{1304}.$$

The values of these moments and thrusts can be plotted to give influence lines which can be used for any type of vertical load on the arch rib. The shears, reactions, etc., may then be determined by simple statics. However, as the effect of a uniformly-distributed load over the whole or half of the span is generally all that is required for design, this is obtained by integration of the values of H_1, etc., for a uniformly-distributed load w as follows:

$$H_1 = \frac{15wL}{163}\int_0^x a(1-a)(5+4a-4a^2)da = \frac{wL}{163}\left(\frac{75a^2}{2} - 5a^3 - 30a^4 + 12a^5\right)_0^x.$$

If the uniformly-distributed load extends horizontally from springing B to the middle of the span, x will be 0·5. For a uniformly-distributed load over the whole span $H_1 = \frac{29wL}{326}$, acting inwards at A and D and outwards at B and C.

In a similar manner integration of the expressions for M_{BA}, etc., gives the following results for a uniformly-distributed load on the span:

$$M_{BA} = \frac{-16wL^2}{326},\ M_{AB} = \frac{-13wL^2}{326},\ M_{CD} = \frac{16wL^2}{326}, \text{ and } M_{DC} = \frac{13wL^2}{326}.$$

Horizontal Loads on the Columns.

Moments M_A and M_B at the fixed ends A and B are caused by the horizontal load (*Fig.* 6). These moments are distributed in *Table* III.

$$LH_1 \text{ at A} = -\frac{18M_B}{35} + M_A + \frac{17M_B}{70} + \left(PbL \text{ or } \frac{p_1L^2}{2}\right),$$

where p_1 is the horizontal load per foot of height of the column.

$$LH_1 \text{ at B} = \frac{19M_B}{70} - M_A + \left[P(1-b)L \text{ or } \frac{p_1L^2}{2}\right],$$

$$LH_a = \frac{5}{8}\frac{L}{r}\left(\frac{18M_B}{35} + \frac{3M_B}{35}\right) = \frac{105M_B}{140}.$$

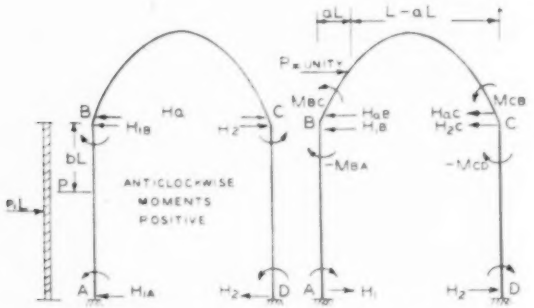


Fig. 6.

Fig. 7.

TABLE III.

	A	½	8 B 9	−½	9 C 8	½	D
	M_A		$-M_B$				
			$^8/_{17}M_B$	$^8/_{17}M_B$			
			ETC	$-^3/_{17}M_B$			
TOTAL	$M_A + {}^{17}/_{70}M_B$		$-{}^{18}/_{35}M_B$	$^{18}/_{35}M_B$	$-^3/_{35}M_B$	$^3/_{35}M_B$	$^3/_{70}M_B$

Then the resultant force which acts on a virtual support at B is

$$P_B = \frac{105M_B}{140L} - \frac{M_A}{L} + \frac{19M_B}{70L} + \left[P(1-b), \text{ or } \frac{p_1L}{2}\right],$$

acting to the left. The force at C is $P_C = \frac{105M_B}{140L} + \frac{18M_B}{140L} = \frac{123M_B}{140L}$, acting to the right.

On removal of the virtual supports at B and C, the components of P_B and P_C act in the opposite direction to the resultant forces due to the resistance of the frame, and these forces are added in the directions shown in *Fig.* 3(*a*) for P_B and in the direction opposite to that shown in *Fig.* 3(*b*) for P_C. The final results are obtained from *Figs.* 6 and 3:

$$H_{1A\,final} = H_{1A} + \frac{221P_B}{326} - \frac{105P_C}{326}$$

$$M_{B\,final} = -\frac{36M_A}{70} + \frac{745P_BL}{2608} - \frac{233P_CL}{2608}.$$

$$H_{1B\,final} = H_{1B} - \frac{221P_B}{326} + \frac{105P_C}{326},$$

$$M_{A\,final} = M_A + \frac{17M_B}{70} + \frac{1023P_BL}{2608} - \frac{607P_CL}{2608}.$$

$$H_{2B\,final} = -H_2 - \frac{105P_B}{326} + \frac{221P_C}{326},$$

$$M_{C\,final} = \frac{6M_B}{70} + \frac{233P_BL}{2608} - \frac{745P_CL}{2608}.$$

At B, $H_{arch} = H_{arch} - \frac{105P_B}{326} - \frac{105P_C}{326}$,

$$M_{D\,final} = \frac{3M_B}{70} + \frac{607P_BL}{2608} - \frac{1023P_CL}{2608}.$$

The usual horizontal load on a frame of this kind is that due to wind and a uniform load of p_1h or p_1L (since h is assumed to be equal to L) gives fixed-end moments $M_A = \frac{p_1L^2}{12} = -M_B$ and causes the following final thrusts and moments:

$$H_{1A} = p_1L\left[\frac{471}{70} + \left(\frac{221}{376} \times \frac{843}{140}\right) - \left(\frac{105}{326} \times \frac{123}{140}\right)\right] = \frac{737p_1L}{840}, \quad M_{BA} = \frac{78{\cdot}5p_1L^2}{840}.$$

$$H_{1B} = p_1L\left[\frac{369}{70} - \left(\frac{221}{320} \times \frac{843}{140}\right)\text{etc.}\right] = \frac{103p_1L}{840}, \quad M_{AB} = \frac{238{\cdot}5p_1L^2}{840}.$$

$$H_2 = -\frac{103p_1L}{840}, \quad M_{CD} = \frac{26p_1L^2}{840}.$$

$$H_{arch}\text{ at B} = \frac{-103p_1L}{840}, \quad M_{DC} = \frac{77p_1L^2}{840}.$$

Check:

$$M_{AB} + M_{BA} - \frac{p_1L^2}{2} = \frac{(317 - 420)p_1L^2}{840} = \frac{-103p_1L^2}{840} = -(M_{CD} + M_{DC}).$$

Horizontal Load acting on the Arch.

The fixed-end moments at B and C due to P (*Fig.* 7) are

$M_B = 2Pa(1-a)^2(2-7a+8a^2)$. $\quad M_C = 2Pa^2(1-a)(3-9a+8a^2)$.

These are distributed in the same manner as the moments in *Table* II and give the same summations so that

$$M_{BA} = -\frac{1}{35}(17M_B + 3M_C). \quad M_{AB} = -\frac{1}{70}(17M_B + 3M_C).$$

$$M_{CD} = -\frac{1}{35}(17M_C + 3M_B). \quad M_{DC} = -\frac{1}{70}(17M_C + 3M_B).$$

Substituting in these expressions the values of M_B and M_C,

$$M_{BA} = -\frac{La(1-a)}{35}(34 - 144a + 228a^2 - 112a^3) = 2M_{AB}.$$

$$M_{CD} = -\frac{La(1-a)}{35}(6 + 24a - 108a^2 + 112a^3) = 2M_{DC}.$$

As $H_{aC} = H_{0C} + \frac{5L}{8r}(M_{BC} - M_{CB})$, and from an article in this journal for April, 1946, $H_{0C} = -\frac{a}{2}(5 - 5a + 10a^2 + 20a^3 - 8a^4)$, and as H_{0C} is independent of r, substituting for M_{BC} and M_{CB},

$$H_{aC} = \frac{a}{14}(21 + 63a - 322a^2 + 420a^3 - 168a^4), \text{ acting inwards at C.}$$

Also $H_{aB} = 1 - H_{aC}$,

$$H_{1B} = \frac{3}{70}(17M_B + 3M_C) = \frac{1}{70}(102a - 534a^2 + 1116a^3 - 1020a^4 + 336a^5),$$

$$H_{2C} = \frac{3}{70}(17M_C + 3M_B) = \frac{1}{70}(18a + 54a^2 - 396a^3 + 660a^4 - 336a^5).$$

The resultant force

$$P_B = H_{1B} + H_{aB} = \frac{1}{70}(70 - 3a - 849a^2 + 2726a^3 - 3120a^4 + 1176a^5),$$

and $P_C = H_{2C} + H_{aC} = \frac{1}{70}(123a + 369a^2 - 2006a^3 + 2760a^4 - 1176a^5).$

These forces are assumed to be resisted by supports at B and C and on removal of these supports the components of P_B and P_C, acting in the opposite direction to the resultant forces, are supplied by the rigidity of the frame. For P_B these forces act in the same direction as those shown in *Fig.* 3(*a*), and for P_C the forces act in the opposite direction to those shown in *Fig.* 3(*b*). Therefore the final thrusts are :

$$H_{1\,final} = H_{1B} - \frac{221P_B}{326} - \frac{105P_C}{326}$$

$$= -\frac{1}{2282}(1547 - 2100a + 2520a^2 + 2800a^3 - 6720a^4 + 2688a^5)$$

acting to the right at B and to the left at A.

$$H_{2\,final} = H_{2C} - \frac{105P_B}{326} - \frac{221P_C}{326}$$

$$= -\frac{1}{2282}(735 + 2100a - 2520a^2 - 2800a^3 + 6720a^4 - 2688a^5)$$

acting to the right at C and to the left at D.

The final moments are

$$M_{BA\,final} = M_{BA} + \frac{745P_BL}{2608} + \frac{233P_CL}{2608}.$$

$$M_{AB\,final} = M_{AB} + \frac{1023P_BL}{2608} + \frac{607P_CL}{2608}.$$

$$M_{CD\,final} = M_{CD} + \frac{745P_CL}{2608} + \frac{233P_BL}{2608}.$$

$$M_{DC\,final} = M_{DC} + \frac{1023P_CL}{2608} + \frac{607P_BL}{2608}.$$

These may be written,

$$M_{BA\,final} = \frac{745L}{2608} - \frac{aL}{4564}(3773 - 9548a + 9422a^2 - 2303a^3 - 448a^4),$$

$$M_{AB\,final} = \frac{1023L}{2608} - \frac{aL}{4564}(427 + 4508a - 15022a^2 + 15743a^3 - 4928a^4),$$

$$M_{CD\,final} = \frac{233L}{2608} + \frac{aL}{4564}(1491 - 420a - 4270a^2 + 4543a^3 - 448a^4),$$

$$M_{DC\,final} = \frac{607L}{2608} + \frac{aL}{4564}(2709 - 4620a - 1330a^2 + 8897a^3 - 4928a^4).$$

The most usual horizontal load is that due to wind. Let p_2 be the unit load due to wind, and p_2s be the load per foot of height of the vertical projection r of the arch rib from springing to crown. Integration, with respect to y, of H_1, H_2, M_{BA}, etc., between $y = r$ and zero gives the following final moments and thrusts:

$$M_{AB} = \frac{13{,}286p_2Lrs}{365{,}120}, \qquad M_{BA} = \frac{77{,}219p_2Lrs}{365{,}120},$$

$$M_{CD} = \frac{48{,}291p_2Lrs}{365{,}120}, \qquad M_{DC} = \frac{106{,}749p_2Lrs}{365{,}120},$$

$$H_{1A} = H_{1B} = \frac{10{,}504p_2rs}{18{,}256}, \qquad H_{2C} = H_{2D} = \frac{7752p_2rs}{18{,}256}.$$

The result may be checked from the condition that the shear in the columns must equal the horizontal load, therefore:

$$\Sigma M = p_2Lrs \text{ and } \Sigma H = p_2rs.$$

To integrate with respect to a the equations for H_1, M_{BA}, etc., must first be multiplied by $dy = 4r(1 - 2a)da$, with the limits of a from 0·5 to zero.

Having calculated the indeterminate moments and thrusts, the moments, shears, reactions, and direct forces at other positions on the frame may be obtained by statics.

(To be continued.)

A Dam at Bort-Les-Orgues, Auvergne, France

In the August, 1952, number of this journal notes were given on the method of concreting the dam at Bort in the Auvergne district of France. This is part of the hydro-electric works which will provide 2400 million kw.-hours a year. *Figs.* 1 and 2 show the downstream face of the dam at Bort with the power-house in process of construction. The dam is of the gravity type, having a total height of 400 ft., a length at the crest of 1300 ft. and a thickness at the base of 280 ft. The work was begun in the year 1947 and is expected to be completed in 1953. A central-mixing plant was built on the site to which two ropeways, 2½ miles long, delivered 200 tons of aggregate per hour. The mixing plant comprised six machines each with a batch capacity of 4 cu. yd. The concrete was distributed to the tower cranes seen in *Fig.* 4 by skips travelling on a

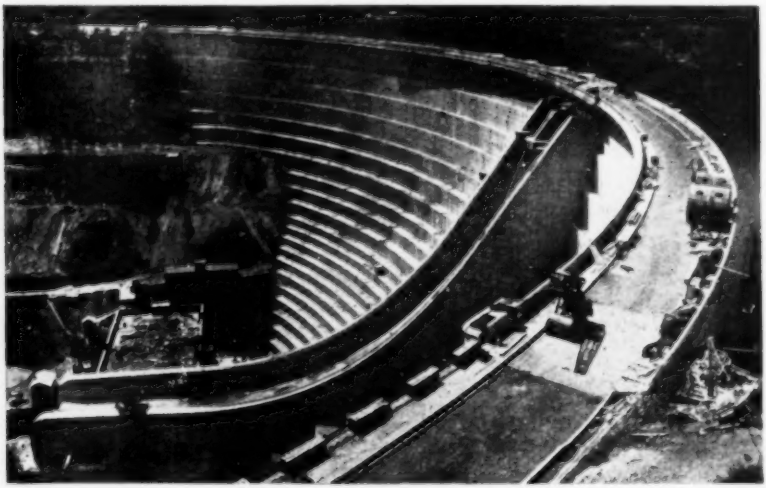


Fig. 1.—The Dam nearing Completion.

Fig. 2.—The Concrete Mixing Plant (being removed) is to the Left.

runway. The concrete was placed in blocks interlocking with one another, contiguous blocks being cast at different times so as to allow of the dissipation of the heat generated by the cement. *Fig.* 5 shows the upstream face with the water at high level. Under test with full water pressure the centre of the dam had a horizontal deflection of 1¼ in.

The pipe (*Fig.* 3), which is at present under construction to bring water from the river Rheu to Bort, has a bore of 16 ft. 6 in. and is about 15 miles in length. It consists of a concrete core surrounded by prestressed steel bars which are given a protective cover of concrete. The method of reinforcing the pipe is to encircle it by separate groups of six steel

Fig. 3.—Prestressed Concrete Pipe 16 ft. 6 in. Diameter.

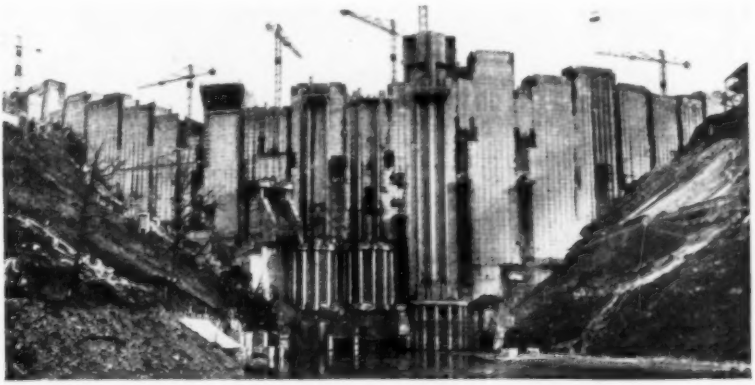


Fig. 4.—Upstream Face During Construction.

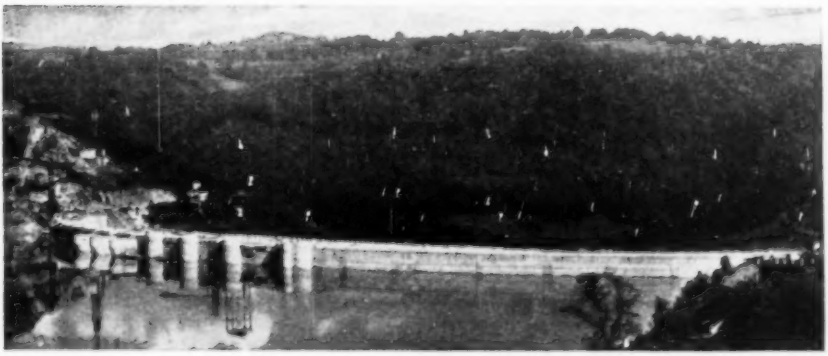


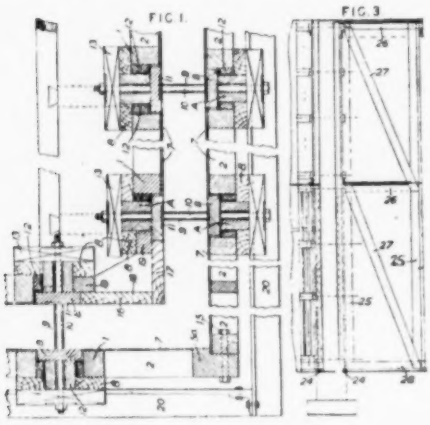
Fig. 5.—Upstream Face.

bars of small diameter kept apart by distance-pieces and having their ends secured to steel plates drilled to receive bolts. Each set of six bars is a little shorter than half the circumference of the core, so that two sets encircle the core. They are tensioned by the steel bolts connecting adjacent pairs of end-plates. Expansion joints are provided at intervals, and are compressed by similar groups of small bars. The dam was constructed by Entreprises Metropolitains et Coloniales.

Patents Relating to Concrete.

Shutters for Walls.

WALL shutters comprise pairs of T-section uprights each consisting of two members (A) with a gap between and a cross member (B). Tie bars (10), with spacing sleeves (9) which abut against plates (11) recessed in the faces of the uprights, pass through the uprights and have turnblocks (13) on their ends retained by nuts or the heads of the bolts. The panels (7) are secured against the rebates of the uprights by wedges (8) driven between the framing



of the panels and the wings of the turnblocks. The turnblocks, which are the same width as, or of less width than, the part of the uprights against which they abut may be turned through 90 deg. after removing the wedges to allow the panels (7) to be removed. The panels are of wire mesh fixed to frames (1, 2), to the vertical edges (1) of which fillets (12) are secured to abut the rebates on the uprights. For external corners one member of the frame is rebated as shown at (5*a*) to receive the adjacent frame, and the two are secured by bolts (15). At internal corners boards (16, 17) are provided, with brackets (18) and fillets (19) to engage the posts. The shutter is erected with the posts at suitable intervals apart on a soleplate (24), the posts on the inside being supported by strutting comprising uprights (25), horizontal members (26) and braces (27) supported on a horizontal framework (28). The shutter is stiffened by steel angles (20) fitted immediately above or below the turnblocks (13), and horizontal bracing members may also be provided secured to the bolts (10) on the outside. Brackets to support scaffold platforms may be secured to the outer posts. Frames may be fastened to the shutter to provide openings for windows or doors.—No. 635,490. Cellular Concrete Ltd., and J. Miller. December 10, 1946.

Porous Concrete.

POROUS concrete is prepared from a cement mixture containing an aqueous solution of polyvinyl alcohol of high molecular weight, air or another gas being incorporated by agitation either in the polyvinyl alcohol solution or in the mixture. In a preferred process a surface-tension reducing agent insensitive to the ingredients of the cement, particularly a sulphonate of a fatty acid of high molecular weight or a sulphonated derivative of alkylated naphthalene, is added to a polyvinyl alcohol solution, which preferably contains 1·5 per cent. or less of polyvinyl alcohol, the solution foamed and a mixture of cement and water subsequently mixed with this foam. The cement mixture may contain fillers such as fine sand or infusorial earth, and the dimensions of the pores of the final product may be regulated by varying the concentration of the polyvinyl alcohol and of the surface-tension reducing agent if present.—No. 633,114.—Soc. Anon. de Diffusion Commerciale et Industrielle. May 14, 1947.

[Publication of patent specifications has been delayed due to the war.]

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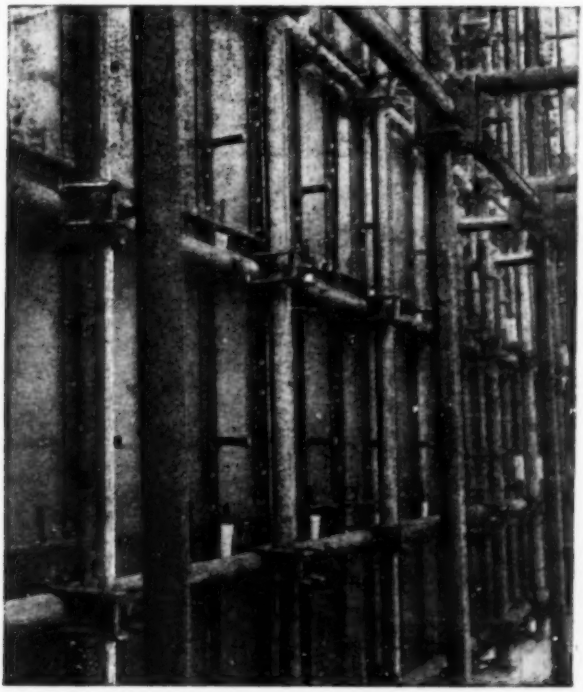

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Prestressed Bridge with a Continuous Girder.

THE bridge shown in *Fig.* 1 is 446 ft. long, comprising end spans of 59 ft. and four intermediate spans of 82 ft. It is prestressed by the system devised by Dr. Leonhardt and described in this journal for November, 1952. The following notes are from "Die Bautechnik" for April, 1953.

The bridge consists of a hollow girder 11 ft. 2 in. wide and varying in depth from 6 ft. 7 in. at the supports to 5 ft. at the midspans. The deck is 1 ft. 2 in. deep between the webs of the girder and extends 3 ft. 4 in. on one side to form a footpath. On the opposite side the projection is 1 ft. and forms a kerb. The webs are 1 ft. 6 in. wide. For a distance of 8 ft. 6 in. on both sides of each support the bottom slab varies in depth from 1 ft. 1 in. at the webs to 10 in. at the centre. For the remainder of each span the soffit slab extends only 2 ft. 6 in. from each web. The girder is stiffened transversely by ribs at intervals of about 19 ft. 6 in.; at the internal supports the ribs are 1 ft. 8 in. wide and the intermediate ribs 10½ in. wide.

Longitudinally each web is prestressed by 320 stranded cables each comprising seven 0·1-in. wires. Each group of 320 cables is contained in a sheath, 6½ in. by 5½ in., and curved upwards over the supports. The cables are in a continuous length and at each end of the bridge they pass around a semi-circular crosshead of concrete (*Fig.* 2) supported on steel plates to allow the crosshead to slide. The tensioning force is applied by jacks which are left in the structure, the jacks comprising concrete pistons sliding in steel cylinders and operated by hydraulic

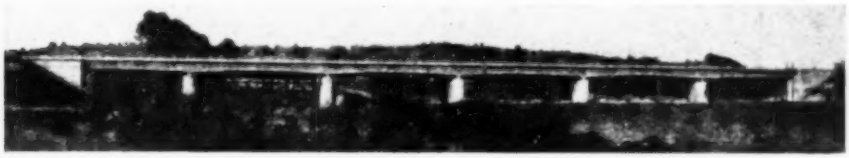

Fig. 1.—Bridge over the River Kocher.

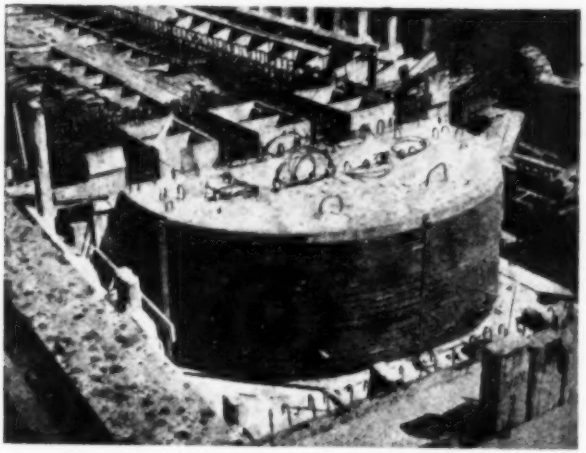

Fig. 2.—One of the Cross-heads.

pressure. The jacks are placed in a gap between the solid ends of the girder and the crosshead. The total force required was 1870 tons and was obtained by simultaneous movement of both crossheads. The force was applied in two increments, the first being 40 per cent. of the total force. This was maintained for about fourteen days so that the shutters could be removed and thus the final tensioning could counteract the stresses due to the dead loads. In order to overcome the losses due to friction the final force was increased to 2000 tons. The deck was prestressed transversely by the Dywidag system using high-tensile bars with threaded ends anchored by nuts and plates.

A Combined Mixer and Batch-Weigher.

THE attachment to the concrete mixer illustrated enables the aggregates to be weighed while they are being placed in the skip. When in the lower position, the skip rests in a cradle from which weigh-beams operate a pointer on a dial mounted on the chassis of the mixer. The method of using the weighing apparatus is to determine first the weight of each material required in a batch, and to set pointers on the dial to indicate the total weights required when the correct quantity of each material is placed in the skip in the specified order.

The weighing device, the level of which is adjusted by built-in jacks, is mounted on a three-point suspension so that the mixer need be only roughly levelled. During weighing the skip is not in contact with the mixer, and rubber joints insulate the dial from the chassis and prevent vibration being transmitted to the weighing mechanism. The cradle is folded up with the skip when the machine is to be transported. The machine is made by Blaw Knox, Ltd., and the weighing device can be used with this firm's 10/7 (as in the illustration) or 14/10 mixers.

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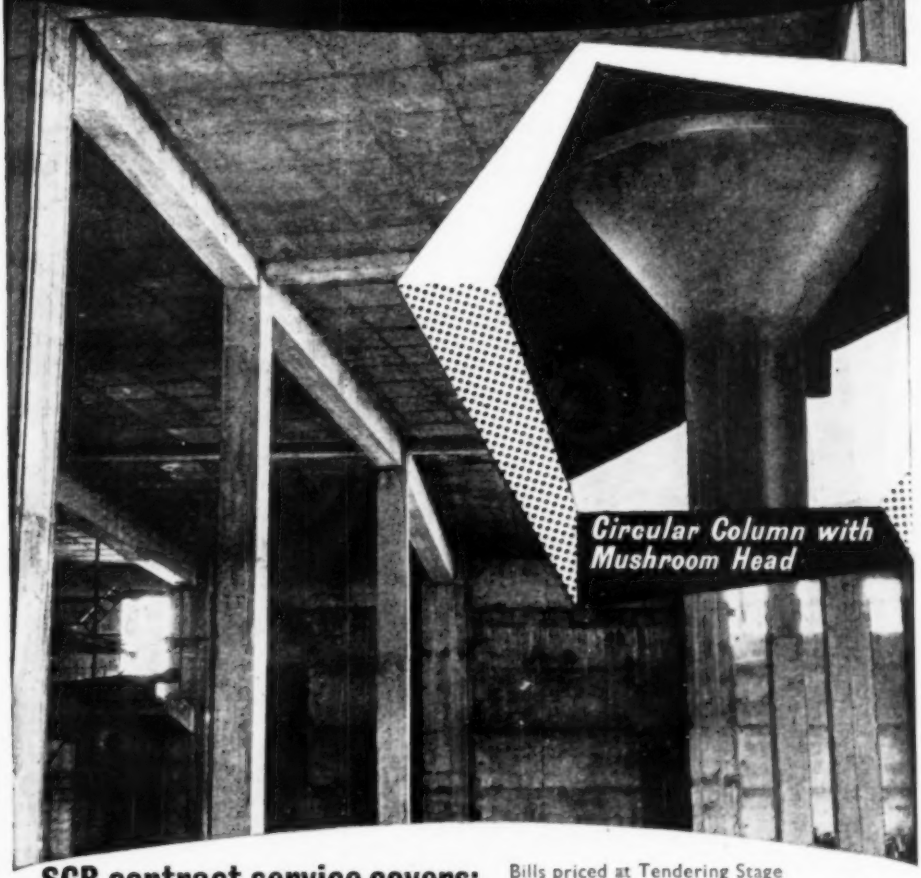

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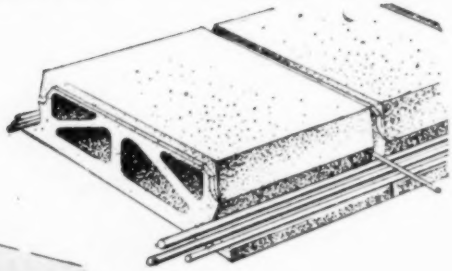
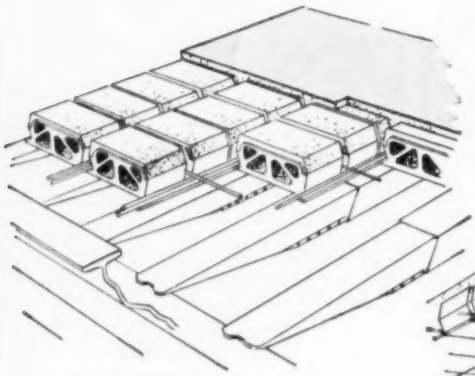
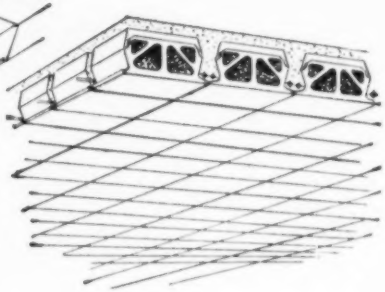




Showing Two-way Reinforcement and Hollow Concrete Blocks laid on Trianco Telescopic Centers.



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A Prestressed Concrete Bridge in Holland.

THE bridge shown in *Fig.* 1 is at Malden, in Holland, and replaces one which was destroyed during the war. The deck slab is 13¾ in. thick by 15 ft. 9 in. wide and is continuous over three spans. The outer spans are each 26 ft. 10 in. long and the central span 35 ft. 1 in. The two intermediate supports are reinforced concrete walls, about 12 in. thick, 18 ft. high, and the full width of the bridge. The walls are hinged at their bases and at the top where they support the slab. One end of the slab rests on a rocker support and the other on a support allowing rotation only (*Figs.* 2 and 3).

The bridge was designed for the loads specified in the Dutch regulations of 1933 for class B roads, that is for a uniformly-distributed load of 82 lb. per square foot plus a vehicle weighing 40 tons; the allowance for impact was 17 per cent. The allowable stresses were 1850 lb. per square inch compression in the concrete and 142,000 lb. per square inch tension in the steel; the ratio of the final stress in the steel to the initial stress was assumed to be 0·85. Under the most adverse distribution of superimposed loads there was a tensile stress of 114 lb. per square inch in the top of the slab of the central span,

Fig. 1.

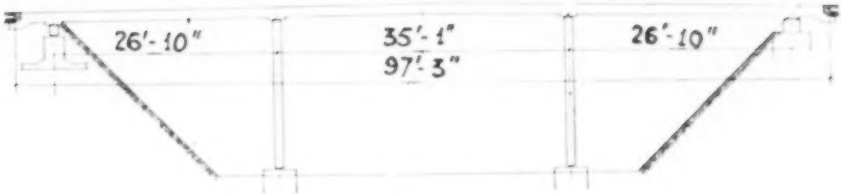


Fig. 2.

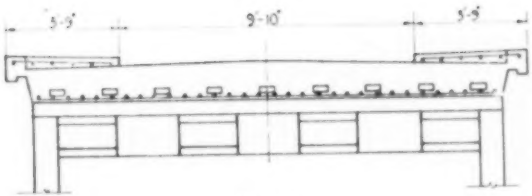


Fig. 3.

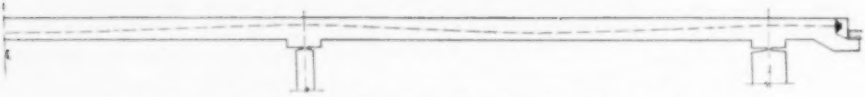


Fig. 4.—Longitudinal Section showing position of Cables.

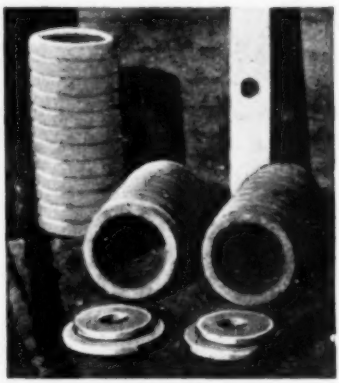
and mild steel reinforcement was added to resist this. There is no transverse prestress, but reinforcement bars were placed in the bottom of the slab across its width. The slab was prestressed by nine cables of the Magnel-Blaton type each containing forty-eight wires of $\frac{1}{8}$ in. diameter. The cables were continuous across the three spans and were laid in the form of a parabola of the fourth degree with an eccentricity at the supports of 2·8 in. above the centre of the slab and an eccentricity at mid-span of 2·5 in. below the centre of the slab (*Fig.* 4).

The cavities for the cables were formed by solid rubber cores of rectangular section and, as these were available in lengths of about 65 ft., the bridge was concreted in two portions, a vertical joint, without special reinforcement, being made at the centre. The concrete was consolidated by immersion vibrators and its strength at twenty-eight days was between 6750 lb. and 7500 lb. per square inch.

It is believed that this is the first prestressed concrete bridge continuous over three spans to be built in Europe. These notes are abstracted from an article by M. G. Baar and M. F. Riessauw in the Belgian magazine "Precontraint" (No. 1, 1952).

Fixing Bolts in Concrete.

A MEANS of positioning bolts in concrete which has been used in the construction of a steel works at Scunthorpe is shown in *Figs.* 1 and 2. The concrete cylinders have internal diameters from 3 in. to 6 in. to accommodate bolts from $\frac{1}{2}$ in. to $2\frac{1}{2}$ in. diameter. The thickness of the wall is from $\frac{1}{2}$ in. to $\frac{5}{8}$ in., and the compressive strength of the concrete is stated to be



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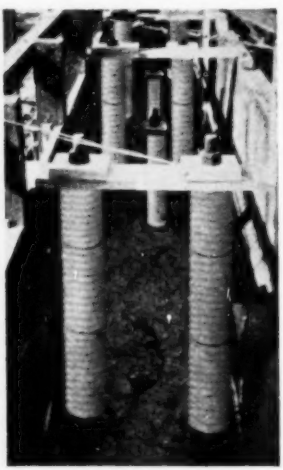
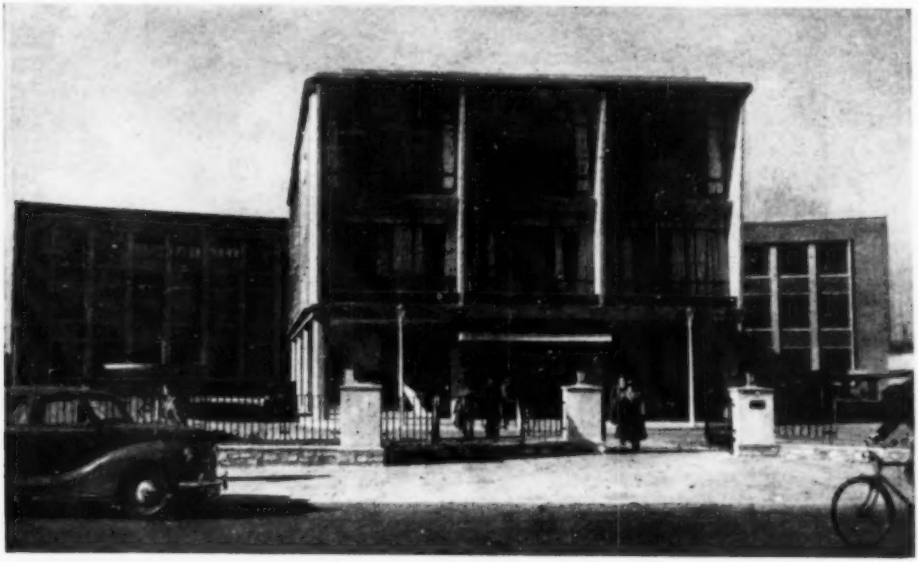


Fig. 1.

1000 lb. per square inch. The cylinders are supplied in lengths to suit the lengths of the bolts to be used. Precast concrete locating pieces, with a rebate at the edge to fit the cylinder, are used at the top and bottom of each cylinder. Both faces of the cylinders are serrated to form a key for the surrounding concrete and also for the grout with which the cylinders are filled. The top locating piece can be recovered and used again. The cylinders and accessories are supplied by Messrs. Leeson, Ltd., of Leeds.



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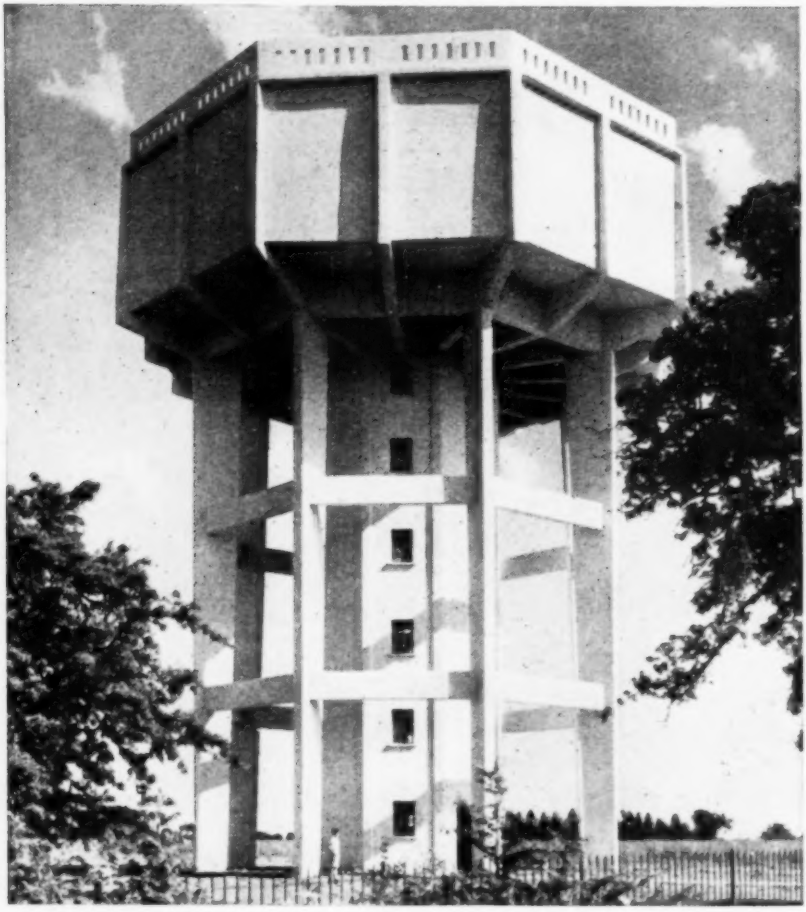

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Sulphate Attack on an Underground Concrete Culvert.

THE following is an extract from a paper by M. Chevrier and M. Houbas published in the twelfth volume of the Publications of the International Association for Bridge and Structural Engineering.

Calcium sulphate in solution combines with the calcium aluminates of hardened cement and forms sulpho-aluminates with accompanying increase of volume. In freshly-mixed mortar these salts produce no apparent expansion while the mortar is wet and may contribute towards hardening, as with metallurgical, super-sulphated, and high-alumina cements. The sulpho-aluminates, when formed by solid salts, develop disintegrating forces. In the case to be described, the calcium sulphate is in solution and reacts with the solid aluminate. Since ordinary Portland cement was used for the culvert, there was no aluminate in solution because of the presence of free lime.

The culvert (*Fig.* 1) conveys water from the river Arc to the hydro-electric power station at Avrieux in the French Alps, and is in a tunnel which was driven through a Triassic rock containing pockets of anhydrite and gypsum. Percolating water dissolves these substances and forms large voids which cause slips and subsidences. Almost as soon as the first stage of the culvert had been completed and lined it was destroyed by ground movements. Good ground was too far away for the line of the culvert to be altered, so an unusual kind of construction was adopted. An egg-shaped reinforced concrete conduit was built inside the heading with about 2 ft. 6 in. clearance between the outside of the conduit and the surface of the rock. The conduit was supported at intervals on concrete pedestals resting on the concrete floor of the tunnel but isolated from it by a few courses of brickwork. In spite of these precautions the conduit was attacked in two ways by sulphate-impregnated water seeping through the rock: (1) The pedestals and the lower parts of the conduit were attacked by the capillary absorption of water from the floor of the tunnel causing complete disintegration of the concrete and corrosion of the reinforcement. (2) The top of the conduit was

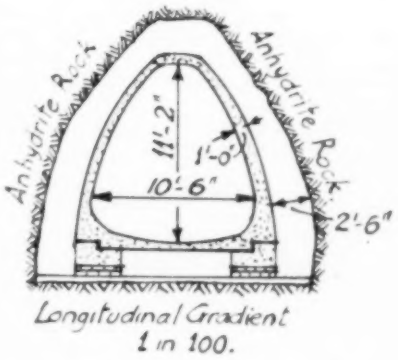

Fig. 1.—Cross Section of Culvert.

attacked by water, dropping from the roof of the tunnel, which formed an efflorescence, from 4 in. to 6 in. thick, over the surface of the concrete, the aggregate being surrounded by a white oily paste. A light scraping sufficed to expose the reinforcement, of which some bars of 1 in. diameter were completely corroded at places.

Repairs are being carried out with high-alumina cement and it is intended, after cleaning and draining, to surround all the lower part of the conduit with slag-cement concrete to a height of 5 ft. from the floor. Slag cement has been chosen rather than super-sulphated cement to avoid chemical reactions at the joints between the old and the new concretes. Super-sulphated cement does not react with other cements, but old Portland cement would be affected by the calcium sulphate (about 15 per cent.) in super-sulphated cement. A hardened super-sulphated cemeat does not harm Portland cement because the calcium sulphate which it contains is fixed in the form of tricalcium sulpho-aluminate which is non-expansive and insoluble. During the period of hardening there is, however, a possibility of reaction between them. Normally there is nothing to fear from the capillary movements from Portland cement to super-sulphated cement. The latter cement, being ground extremely finely, forms capillaries in the

concrete which are narrower than those in Portland cement concrete; moreover it absorbs much water whilst hardening. However, if as a result of unfavourable conditions, such as the super-sulphated cement concrete being too permeable and, being under hydrostatic pressure, movement may take place in the opposite direction. The dampness in the tunnel assists the hardening of this cement, and its strength need not be great as it will not be heavily loaded.

Prestressed Concrete Pipes in Kenya.

WORK has commenced on the construction of a factory at Voi, Kenya, for the production of prestressed concrete pipes for the Mombassa water scheme. The pipe-line will be 155 miles long. The pipes vary in diameter generally from 1 ft. 6 in. to 2 ft. 6 in. and will be in lengths of 21 ft. The pipes will be made by Société des Tuyaux Bonna and laid by Société Dumez.

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